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Storm and Combined Sewer Pollution Sources and Abatement

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Six urban drainage basins ~~within the City of~~ in Atlanta, Georgia, served by combined and separate sewers, ~~are~~ were studied to determine the major pollution sources during storm events. Rainfall frequency analysis and simulation techniques ~~are~~ were utilized to obtain design criteria for alternative pollution abatement schemes.

High frequency storms ~~are shown to~~ cause the worst impact and most of the pollution from combined sewer areas. Annual BOD from these areas is 2,078,000 pounds, or 460 lbs/acre, of which 57 percent is due to storms of two–week or higher frequency. Bypassing of wastewater treatment plant flows during storms adds 690,000 pounds BOD/year. Runoff from storm–sewered areas, at 253 lbs/acre, adds 5,577,000 pounds/year. Overflows and bypassed flows have severe impact upon the South River, due to their high deoxygenation rates and coliform concentrations.

Annual BOD reduction from combined sewer areas of 57 percent may be achieved for a total annual cost of $165,000, by modifying the three regulators and treating 80 percent of the overflows, in conjunction with storage sufficient to contain a two–week storm. Alternate, less favorable solutions include storage and treatment at existing treatment plants, and storage with release to receiving streams after chlorination. Separation of combined sewers would achieve 60 percent BOD removal for $3,030,000/year.

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STORM AND COMBINED SEWER
POLLUTION SOURCES AND ABATEMENT

ATLANTA, GEORGIA

for the

ENVIRONMENTAL PROTECTION AGENCY
WATER QUALITY OFFICE

by

Black, Crow and Eidsness, Inc.
Consulting Engineers
Atlanta, Georgia

Program Number 11024 ELB

Contract No. 14-12-458

January, 1971

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This report has been reviewed by the Environmental Protection Agency Water Quality Office and approved for publication. Approval does not signify that the contents necessarily reflect the views and policies of the Environmental Protection Agency, nor does mention of trade names or commercial products constitute endorsement or recommendation for use.

ABSTRACT

Six urban drainage basins within the City of Atlanta, Georgia, served by combined and separate sewers, are studied to determine the major pollution sources during storm events. Rainfall frequency analysis and simulation techniques are utilized to obtain design criteria for alternative pollution abatement schemes.

High frequency storms are shown to cause the worst impact and most of the pollution from combined sewer areas. Annual BOD from these areas is 2,078,000 pounds, or 460 lbs/acre, of which 57 percent is due to storms of two–week or higher frequency. Bypassing of wastewater treatment plant flows during storms adds 690,000 pounds BOD/year. Runoff from storm–sewered areas, at 253 lbs/acre, adds 5,577,000 pounds/year. Overflows and bypassed flows have severe impact upon the South River, due to their high deoxygenation rates and coliform concentrations.

Annual BOD reduction from combined sewer areas of 57 percent may be achieved for a total annual cost of $165,000, by modifying the three regulators and treating 80 percent of the overflows, in conjunction with storage sufficient to contain a two–week storm. Alternate, less favorable solutions include storage and treatment at existing treatment plants, and storage with release to receiving streams after chlorination. Separation of combined sewers would achieve 60 percent BOD removal for $3,030,000/year.

This report is submitted in fulfillment of Contract 14–12–458 between the Environmental Protection Agency and Black, Crow and Eidsness, Incorporated.

Key Words: Storm overflow, combined sewer overflow, simulation, frequency analysis, treatment costs, economic analysis, land use indicators, unit hydrographs.

Seis cuencas urbanas de drenaje en la ciudad de Atlanta, Georgia, servidas por cloacas combinadas y separadas, han sido estudiadas para determinar las principales fuentes de contaminación durante las ocurrencias de lluvia. Elanálisis de frecuencia de lluvia y métodos de simulación fueron utilizados para obtener criterios de diseño para diferentes métodos de controlar la contaminación.

Se descubrió que las tormentas de frecuente ocurrencia causan el peor impacto y contribuyen la mayor parte de la contaminación en las regiones con cloacas combinadas. El aporte anual de demanda bioquímica de oxígeno (DBO) de estas regiones es 943,000 kg, ó 516 kg/hectárea, de la cual el 57 por ciento es debido a tormentas confrecuencia de dos semanas, ó mayor. El desvio de los flujos en las plantas de puradoras durante las tormentas añade 313,000 kg de DBO por año. El escurrimiento de áreas con drenaje pluvial, que contiene 284 kg/hectárea, añade 2,532,000 kg por año. Los rebosos y desvíos producen un impacto severo sobre el rio South, debido a sus altos coeficientes de consumo de oxígeno y concentraciones de coliformes.

Una reducción del 57 por ciento del DBO proveniente de áreas con cloacas combinadas puede lograrse por un costo total anual de $165,000, modificando los tres reguladores y sometiendo a tratamiento el 80 por ciento de los rebosos, mediante embalse de un volumen suficiente para una tormenta de dos semanas de frecuencia. Otras soluciones no tan favorables incluyen embalse y depuración en plantas existentes; también embalse con aplicación de cloro, previa a la descarga de las aguas al río. La separación de las cloacas combinadas logroría una reducción del 60 por ciento del DBO, a un costo de $3,030,000 anual.

Este informe se rinde en cumplimiento del Contrato 14–12–458 entre el Environmental Protection Agency y Black, Crow and Eidsness, Inc.

Sechs städtische Entwässerungsbecken innerhalb der Stadt Atlanta (USA Staat Georgia), die von verbundenen und einzelnen Abzugskanäle gespeist sind, wurden untersucht, um die Hauptverunreinigungsquellen während eines Unwetters festzustellen.

Analyse der Häufigkeit des Regens, sowie Nachahmungsmethoden wurden gebraucht, um Anlage Kriterien für alternative verunreinigungsverminderung Schemen zu erreichen.

Es liess sich erkennen, dass oftmalige Unwetter den schlimmsten Aufschlag sowie die meisten Verunreinigungen vo verbundenen Abzugskanalgebieten verursachten. Jährlicher Biochemische Sauerstoff Bedarf (BSB) von diesen zwei Gebieten beträgt 943,000 kg., oder 516 kg./ha. Zu den Unwetter, die alle vierzehn Tage oder häufiger vorkommen, ist 57 Prozent davon zurückzuführen. Ausweichung der Fluten der Abwasserbehandlungsstelle während des Unwetters fügt weitere 313,000 kg. BSB/Jahr hinzu. Ablauf von den Gebieten, die mit Abzugskanäle ausgerüstet sind, (284 kg./ha.) fügt weitere 2,532,000 kg./Jahr hinzu. Überflutungen und Ausweichungen haben schlimme Wirkung auf den South River, da sie hohe Entoxydierungsgeschwindigkeiten, sowie hohe Kolonbazillen Konzentrationen besitzen.

Jährliche BSB Verminderung von verbundenen Abzugskanalgebieten mit 57 Prozent könnten mit einem jährlichen Kosten von $165,000 erreicht werden, indem die drei Regler modifiziert sein werden, ferner, 80 Prozent der Überflutung bearbeitet wird, und dazu Lagerung genügend, ein Umwetter von vierzehn Tagen Dauer zu enthalten.

Alternative, jedoch weniger günstige lösungen: Lagerung und Bearbeitung bei den noch verhandenen Behandlungsstellen, und Lagerung mit Abfluss in Flüsse nach chlorinierung. Trennung der verbundenen Kanäle könnte 60 Prozent BSB Beseitigung bei einer Kosten von $3,030,000/Jahr erreichen.

Dieser Bericht wird gemäss Vertrag 14–12–458 zwischen der Umgebungsschutzagentur und Black, Crow und Eidsness, E.V. übergeben.

Six bassins de drainage urbains dans la Ville d'Atlanta, Georgie, servis des égouts combinés et séparés, sont étudiés pour déterminer les sources majeures de la pollution pendant les évènements d'orage. L'analyse de la fréquence de précipitation et les techniques de simulation sont utilisées pour obtenir des critères à dessein pour des plans alternatifs à diminuer la pollution.

Il est montré que les orages en grande fréquence cause le plus grand impact et la plupart de la pollution qui vient des égouts combinés. Il y a une EBO (exigence biochimique d'oxygène) annuelle dans ces regions de 943,000 kg. ou de 460 kg/hectare dont 57 pourcent est dû aux orages à fréquence de deux semaines ou plus rapprochée. Pendant les orages l'évitement des débits venant des installations qui traitent la perte–eau ajoute 313,000 kg. EBO/an. Le débit venant des regions aux égouts à orage, à 284 kg/hectare, ajoute 2,532,000 kg/an. Les débordements et les débits dérivés ont un impact sévère sur le South River à cause de hautes raisons de désoxygénation et de concentrations coliformes.

Une réduction annuelle de EBO peut être achevée pour une somme annuelle de $165,000. en modifiant les trois régulateurs et en traitant 80 pourcent du débordement conjointement avec l'emmagasinage suffisant pour contenir un orage de deux semaines. Des solutions alternatives mais moins favorables comprennent l'emmagasinage et le traitement aux installations existantes et l'emmagasinage avec l'échappement aux rivières de réception après la chloruration. La séparation des égouts combinés accomplirait l'enlèvement de 60 pourcent EBO pour $3,030,000/an.

Ce rapport est présenté pour remplir le contrat 14–12–458 entre le Environmental Protection Agency et Black, Crow and Eidsness, Inc.

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CONCLUSIONS

1. High frequency storms cause significantly greater total annual pollution from combined sewer overflows than low frequency storms. Provision for storage of overflows from a small storm of two–week recurrence interval will enable treatment of 80 to 88 percent of the total annual BOD released in combined sewer overflows in the South River Basin, City of Atlanta.

2. Total annual BOD from combined sewer overflows and intercepted flows amounts to 2,767,000 pounds. Of this amount, 2,078,000 pounds occurs as combined sewer overflow, with an average yield of 460 pounds/acre. The remainder is due to bypassing of wastewater treatment plant flows as a result of the deleterious effect of intercepted combined flows upon influent quality.

3. The impact upon the South River of pollution from individual high frequency storms is more severe than that from storms of low frequency. A two–week storm causes anaerobic conditions in the South River at Klondike Bridge, approximately 19 miles below the study area. A one–year storm lowers dissolved oxygen (DO) levels to one mg/l at this point. Extended duration, low intensity rainfall causes DO to vary around three mg/l.

4. The primary sources of pollution during storm events over the study area, in order of decreasing total annual BOD load, are as follows:

		lbs BOD/year
A.	Storm drainage from urban areas (22,042 acres)	5,577,000
B.	Combined sewer overflows to Intrenchment Creek (3,550 acres)	1,633,000
C.	Bypassing of flows from Intrenchment Creek interceptor	506,000
D.	Combined sewer overflow at McDaniel Street (968 acres)	445,000
E.	Intrenchment Creek wastewater treatment plant effluent	185,000

		lbs BOD/year
F.	Bypassing of flows from South River wastewater treatment plant	183,000
G.	South River wastewater treatment plant effluent	146,000

Of these sources, combined sewer overflows and bypassed flows have the greatest impact upon the South River, due to their high deoxygenation rates and high coliform concentrations. Calculations indicate that such bypassing of all treatment plant flows during a two–week storm occurring over the Atlanta urban area, will cause anaerobic conditions to occur in the South River at Snapfinger Creek. Prevention of bypassing during the same storm will cause the DO profile to reach a deficit of 7.5 mg/l further downstream at Flat Bridge, with the critical point falling outside the South River Basin.

5. The impact upon the South River of bypassing intercepted storm flow at the wastewater treatment plants is far more severe than release of the same flow as combined sewer overflow at the point of interception. The difference in impact is due to the large amount of untreated sewage entering the interceptor from areas served by separate sewer systems. In addition to storm flows, the Intrenchment Creek interceptor carries sewage from 46 percent of the area contributing to the Intrenchment Creek wastewater treatment plant. Bypassing of all flow occurs due to the grit, grease and mud introduced by intercepted combined flows.

6. Wastewater treatment plant effluents during dry weather flow are the primary sources of pollution, causing significant impact upon stream water quality. Combined effluent flows from the two treatment plants average 44 cfs and constitute approximately 75 percent of the streamflow at Bouldercrest. Under these conditions, the critical DO concentration occurs near Panthersville Road, yielding an average deficit of 4.5 mg/l.

7. The average annual BOD from storm sewer areas amounts to 55 percent of that from combined sewer areas. A one–year storm of two–hour duration yields 4.744 and 3.940 lbs BOD/acre/hour of rainfall respectively, for combined and storm sewer areas. By contrast, a two–week storm of five–hour duration yields 0.694 and 0.334 lbs BOD/acre/hour of rainfall, respectively.

8. Twenty–five percent reduction in annual pollution from combined sewer areas may be achieved by modifying regulators to eliminate interception of storm flows. Estimated total cost of this approach is $50,000. Further pollution reduction may be achieved by constructing reservoirs at the overflow sites to store overflows from all two–week storms and major fractions of larger storms.

Treatment of the overflows by screening, dissolved air flotation and chlorination will enable 57 percent reduction in total annual BOD from overflows and bypassed flows. Total annual cost is estimated at $161,000.

9. Treatment of storm runoff from areas served by separate sewer systems was found to be infeasible. The benefits and economics of several feasible alternatives for pollution abatement are shown below, in order of increasing cost–benefit ratio:

	Annual BOD Removed		Cost–Benefit Ratio
Alternative	**lbs**	**Percent**	**$/lb BOD**
Regulator Modification			
Intrenchment Creek	506,000	18	0.0057
McDaniel Street	183,000	7	0.0074
Storage, Screening, Flotation, Cl_2			
Intrenchment Creek	692,000	25	0.17
McDaniel Street	189,000	7	0.19
Storage, Diversion to WWTP			
McDaniel Street	48,000	2	0.23
Storage, Chlorination			
Intrenchment Creek	256,000	9	0.31
McDaniel Street	70,000	3	0.29
Sewer Separation	1,665,000	60	1.82

10. Correlation of pollution load from combined sewers with detailed land use is unrealistic for all but very low frequency storms, in an industrial area. Correlation with population density is significant. Percentage of undeveloped land may also be significant. Pollution from storm sewer areas, and combined sewer areas at very high flow rates, may be correlated with land use.

11. For small urban areas such as those considered in this study, continuous monitoring of rainfall and runoff should be undertaken in such a way as to enable accurate determination of short time interval variations. One–day recording

charts proved sufficient in this study. Weekly charts permit accurate reading of hourly variations only, which is not adequate to define unit hydrograph characteristics.

12. For the subbasins considered in this study, 15–minute records of rainfall and runoff were sufficient to define the unit hydrographs. Higher frequency records, such as five–minute readings, are desirable since they permit accurate assessment of synchronization of rainfall and runoff records as well as better definition of unit hydrograph characteristics.

13. Derivation of unit hydrographs for a number of small urban subbasins requires one rainfall monitoring station for each subbasin. A single rain gage for all subbasins is not sufficient since travel time of a storm across the whole area may be relatively slow by comparison with the concentration time of each subbasin. This causes records at some stations to indicate the presence of runoff prior to onset of rainfall. Accurate synchronization of rainfall and runoff records is most important for derivation of average unit hydrographs.

14. Infiltration to the combined sewer system is not a significant problem in the Atlanta area, due to the abundance of clays and other impervious materials.

15. Utilization of scow floats to measure interceptor flows is successful only when such flows do not surcharge the interceptor, causing the water level to rise in the manhole and the scow float to strike the top of the conduit.

16. Direct stage measurement of storm overflows using stilling wells and water level recorders is limited by very high flow velocities accompanied by large suspended and floating materials. These devices must be located in protected areas with lower flow velocities.

17. Samples collected by automatic sampling devices tend to freeze in the sampling tubes during extremely cold weather. Furthermore, the location of these vacuum–operated devices at safe heights above expected peak flow levels limits the volume of samples that may be collected for analysis.

18. The automatic triggering device utilized during this study is not reliable. Dampness deteriorates electrical contacts and solenoids, causing failure of apparently well insulated parts. The consequent necessity for manual triggering of the automatic samplers reduces their usefulness and indicates the need for an improved triggering device.

19. No significant differences exist between water quality analyses of simultaneous samples obtained by grab and automatic sampling techniques. However, grab sampling enables detailed coverage of the initial few minutes of overflows, during

which variations in water quality are significant. Automatic sampling is more suitable for longer time intervals, in order to cover an entire period of overflow.

20. A trend of decreasing rainfall in a southwesterly direction may exist in the Atlanta area.

21. The COD/BOD ratio of storm water is higher than that for combined sewage. Values found in this study were 3.60 and 2.07 respectively. However, the average COD at the three combined sewer stations was 297 mg/l, compared to 60 mg/l for the storm sewers.

22. The six months originally scheduled for data acquisition in this project proved to be insufficient. After development of accurate rating curves, evaluation of water quality data was possible. However, the evaluation indicated the need for improvement of data in certain areas, in order to obtain statistical validity of resulting relationships. Fourteen months of data acquisition were required.

RECOMMENDATIONS

1. The City of Atlanta should modify existing regulators at the Confederate Avenue, Boulevard and McDaniel Street overflows at a cost of $50,000. Automatic gates at these points will eliminate interception of combined flows and subsequent bypassing of waste treatment plant flows, thereby reducing by 25 percent the annual BOD released to the South River due to combined sewers.

2. Further study should be undertaken to determine settling characteristics, chlorine demand and deoxygenation rates of combined sewer overflows at the two proposed reservoir sites. This will enable more accurate assessment of BOD removal due to settling in the reservoirs, the need for oxygenation of the effluents from the reservoirs, and the required chlorine dosage.

3. Following reevaluation of the feasible treatment alternatives, consideration should be given to implementing that combination of treatment processes yielding the greatest reduction in total annual BOD at the lowest cost–benefit ratio. If the lab studies confirm accuracy of the assumptions underlying the present economic analysis, this combination will include storage, screening, dissolved–air flotation and chlorination of overflows at the McDaniel Street and Intrenchment Creek reservoir sites at a total annual cost of $161,000. When combined with regulator modification, it would yield 57 percent removal of total annual BOD from combined sewer overflows and bypassed flows.

4. Design of facilities to store and treat combined sewer overflows should utilize a high–frequency design storm. This is in direct contrast to present practice, which utilizes 10–15 year design storms at considerable, unnecessary expense. A design storm of two–week recurrence interval, in conjunction with treatment facilities designed to empty the reservoirs within 10 hours of overflow cessation, will enable treatment of 80 percent of total annual BOD from combined sewer overflows in the South River Basin, City of Atlanta.

5. Whenever storage of combined sewer overflows from high frequency storms is feasible, consideration should be given to the possibility of precluding interception of combined flows, and treating all stored overflows at the reservoir site, as recommended in this study. When storage near the overflow site is not feasible, consideration should be given to two alternatives:

    A. Locate treatment facilities at the overflow site to treat the relatively small flows from high frequency storms. Capacity of interceptors would be restricted in order to avoid hydraulic overloading of sewage treatment facilities.

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B. Increase interceptor capacity to enable storage and treatment of overflows from high frequency storms at the existing treatment plant site. Additional facilities would be required to handle the tremendous volumes of flow.

Consideration of these alternatives and of the pollution loading and impact of high frequency storms will permit the most effective management of physical and financial resources available for combatting pollution from urban areas.

6. Further studies of combined sewer pollution should ensure synchronization of all timing mechanisms on rainfall and water level recorders. In urban areas of the size utilized in this study, 15–minute rainfall and runoff observations are adequate. However, five–minute intervals are preferable in order to obtain unit hydrographs that reflect accurately the response of the areas to rainfall. Hardware and computation costs rise sharply with increasing sampling frequency.

7. Automatic triggering devices should be improved in order to fully utilize the advantages of automatic samplers. Either better insulation of electrical components or utilization of a purely mechanical device may increase reliability.

8. Automatic samplers should be improved to prevent freezing of samples in the lines during cold weather. Continuous pumping of a small volume of the flow to be sampled may alleviate this problem.

9. The simulation procedure followed in this study should be applied to other areas of the United States. It offers much greater efficiency of data collection and utilization than has been achieved in previous studies. It also yields insight into the effects of a full range of rainfall characteristics upon pollution from combined sewers.

10. Subsequent studies following the simulation approach of this report should utilize both grab and automatic sampling techniques, in order to relate water quality to flow and time. Coverage of the entire period of runoff requires long sampling intervals, whereas coverage of the initial overflow and peak flows requires sampling at short intervals. Grab sampling is most suitable for such short time intervals.

11. Correlation of pollution load from combined sewers with population density in an industrial area could be improved by consideration of population equivalent of industrial wastes.

12. Minor additional study should be undertaken to determine the unit hydrographs for areas contributing to the Confederate Avenue, Harlan Drive and Federal Prison Branch monitoring stations. This would permit utilization of existing water

quality data at these stations, thereby doubling the number of areas considered in the correlation analysis of pollution load and land uses. Monitoring intervals at these stations were too long to enable calculation of adequate unit hydrographs for this study.

13. Periods of data acquisition on such projects as this should be of sufficient length to enable accurate determination of rating curves. Concurrent acquisition of water quality data may be conducted; however, any proposed schedule should recognize that after completion of the rating curves, analysis of the relationships between water quality parameters and flow rate will probably reveal additional areas of required water quality data. Advance consideration of this need will enable substantial improvement in statistical validity of these relationships.

14. In conjunction with other communities utilizing the South River, the City of Atlanta should determine a point below which water quality will be maintained for a purpose higher than effluent disposal. The potential demand for this water resource for recreation and other uses should be considered prior to establishment of the point.

15. Quality of the effluent from existing wastewater treatment plants should be upgraded as necessary to meet the water quality standards at the established point. Downstream sources of raw waste should be treated regardless of water quality standards, although it is recognized that these municipalities are not within jurisdiction of the City of Atlanta.

SECTION I

INTRODUCTION

Scope and Purpose

The present project covers an engineering investigation and comprehensive technical study to evaluate principal pollution sources within the upper South River basin in Atlanta, Georgia, and to compare alternate solutions and remedial measures to the existing situation. Correlation of pollutional load data and of storm runoff quantity and quality to zoning and land use is also included in the purpose of the study.

Recognized pollution sources contributing to present conditions in the stream are overflows from combined sewer systems; storm runoff from urban and other areas, eventual bypasses of untreated wastes at wastewater treatment plants; and continuous discharges of treated wastes in volumes too large for the assimilative capacity of the receiving stream.

A further purpose of the study is an evaluation of the benefits, economics and feasibility of alternate schemes for control of pollution. This evaluation, although specific for the study area, may be applicable to other cities with similar conditions.

General Background of the Project

Combined sewer overflows have been long recognized as a major source of water pollution. However, until very recently, not too much work had been directed to quantitative investigation and assessment of their pollutional significance. A large fraction of the population, concentrated in the older and most populous metropolitan areas, is served by combined sewer systems. Separation of sanitary and storm systems in these areas would represent an almost insurmountable problem for the Nation.

In addition, recent investigations of characteristics of storm runoff from urban areas have demonstrated that these discharges also constitute a large source of pollution for receiving streams. Therefore, complete sewer separation would not fully protect these waters from urban pollution.

The South River basin in Atlanta, Georgia, is a notorious example of stream deterioration on account of pollution originating from a number of sources deriving from metropolitan growth within the basin. Since the upper boundaries of the basin lie entirely within the City, all flow from the headwaters of the stream is polluted at its origin. Part of the basin is served by combined sewers, ending at major overflow points. Existing open streams originate at these overflow points or at ends of storm sewers. Water supply for the area comes from the Chattahoochee River basin, and reaches the South River only after being used and discharged as waste.

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As an intrastate stream, the South River falls under jurisdiction of the State Water Quality Control Board, which requires secondary treatment of all municipal wastes. At present the upper stream reaches are used solely for the transport of wastewater treatment plant effluent, which constitutes a significant fraction of the dry weather stream flow. In previous years expanding development of the study area has caused conversion of sections of the headwater channels to closed culverts. The trend is expected to continue in the future.

Investigation Procedure

The study included field surveying and mapping of portions of the storm and combined sewer system. Continuous monitoring stations were located at five points within the area to determine volume and distribution of rainfall. Six subbasins were chosen for detailed study, three of which were combined–sewer areas. Overflows and intercepted flows from these three areas were recorded and monitored for various water quality parameters. Runoff from the remaining three storm–sewer areas was also recorded and monitored. Additional flow recording stations were located on Intrenchment Creek and at four points along the South River. Samples were taken periodically at these stations for water quality analysis. Data were collected from January, 1969, until April, 1970.

The IBM 360 computer was utilized extensively throughout the study. Factors affecting concentrations of various water quality parameters were determined, and equations were developed from the data to predict these concentrations during periods of storm runoff. Unit hydrographs relating rainfall to runoff in each subbasin were developed. Existing rainfall records in the Atlanta area were utilized to calculate intensity–duration–frequency curves for storms with return periods ranging from two weeks to one year, since preliminary analysis of the data indicated that the small, high frequency storms contribute significantly to the total annual pollution load upon the South River. A series of simulated rainstorms, typical of those occurring in Atlanta, was created from these curves. For each storm the unit hydrograph calculated from the data was utilized to determine the variation of flow rate with time at the monitoring point. The equations relating BOD concentration to time and flow rate were then utilized to determine the pollution load resulting from each simulated storm, in terms of pounds BOD. The series of simulated storms covered a full range of frequencies and durations. The results were analyzed to establish the relative pollution impact of storm sewers, combined sewers, and wastewater treatment plant effluents and bypasses. Also analyzed was the effect of land uses and storm characteristics upon pollution loads and water quality in the South River. Based upon the results, various pollution abatement schemes were considered, to determine the most effective approach to alleviating the quantity and impact of pollution from urban areas served in part by combined sewers.

SECTION II

DESCRIPTION OF THE STUDY AREA

General

The City of Atlanta, capital of Georgia, is the center of a fast growing metropolitan area with a population presently exceeding 1.3 million. It is located on the Piedmont Plateau, at the foot of the Blue Ridge Mountains, in the north central part of the State. Because of its location, the terrain is rolling to hilly and several ridges cross the area. A subcontinental divide cuts through the heart of the City, separating drainage into two major river basins; namely, the Apalachicola, including the Chattahoochee and Flint rivers, that is tributary to the Gulf of Mexico, and the Altamaha, discharging into the Atlantic Ocean and entirely located within the State of Georgia (see Figure 1).

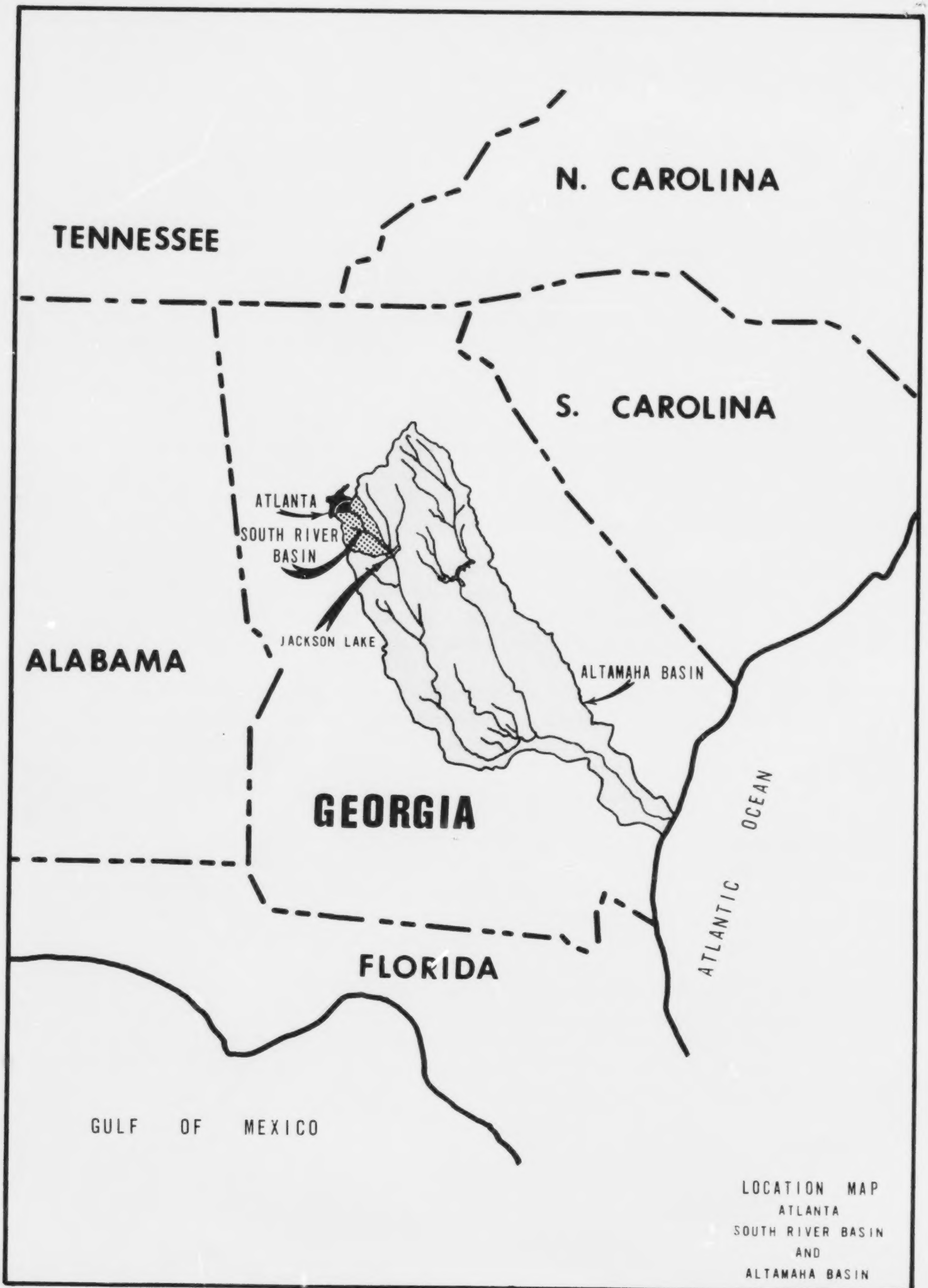
The South River is located within the Altamaha basin, its headwaters reaching downtown Atlanta. This river runs approximately southeast for about 62 miles and ends at Jackson Lake, northeast of the City of Jackson, Butts County. The lake also receives contributions from the Yellow and Alcovy Rivers. Waters from Jackson Lake form the Ocmulgee River that joins the Oconee to form the Altamaha.

The upper South River basin has well defined drainage boundaries consisting of natural ridges, most of them utilized by the numerous railroad companies that serve the Atlanta area as the most convenient routing for their multiple tracks. Elevation of these ridges generally reaches or slightly exceeds 1,000 feet above sea level.

The study area lies entirely within the upper South River basin, and includes parts of Fulton, Clayton and DeKalb Counties. Boundaries of this area were defined in the Plan of Operation for the study. It encompasses a total of 41.5 square miles, covering all the drainage area of the river down to the Bouldercrest Road bridge. Conditions in the river were also monitored at Klondike Road bridge, about 16 miles downstream of Bouldercrest.

Approximately 33 square miles of the upper South River basin fall within the city limits of Atlanta. Topographical information and storm and combined sewer records were lacking or incomplete for this area. As a part of this contract, all elements of these sewer systems were located in the field and plotted in detail on topographic maps previously prepared for the City. Mapped area, city limits and basin boundaries are indicated in Figure 2.

Headwaters of the basin include, to the west and south, additional areas outside of Atlanta proper. Flow contributions enter the Atlanta area from parts of the basin located within the city limits of East Point and Hapeville, to the west, and within Mountain View and Forest Park to the South. Unincorporated areas of Clayton County are also located at the headwaters of the basin south of corporate Atlanta.


N. CAROLINA
TENNESSEE
S. CAROLINA
ATLANTA
SOUTH RIVER BASIN
JACKSON LAKE
ALABAMA
ALTAMAHA BASIN
GEORGIA
ATLANTIC OCEAN
FLORIDA
GULF OF MEXICO

LOCATION MAP
ATLANTA
SOUTH RIVER BASIN
AND
ALTAMAHA BASIN

Figure 1

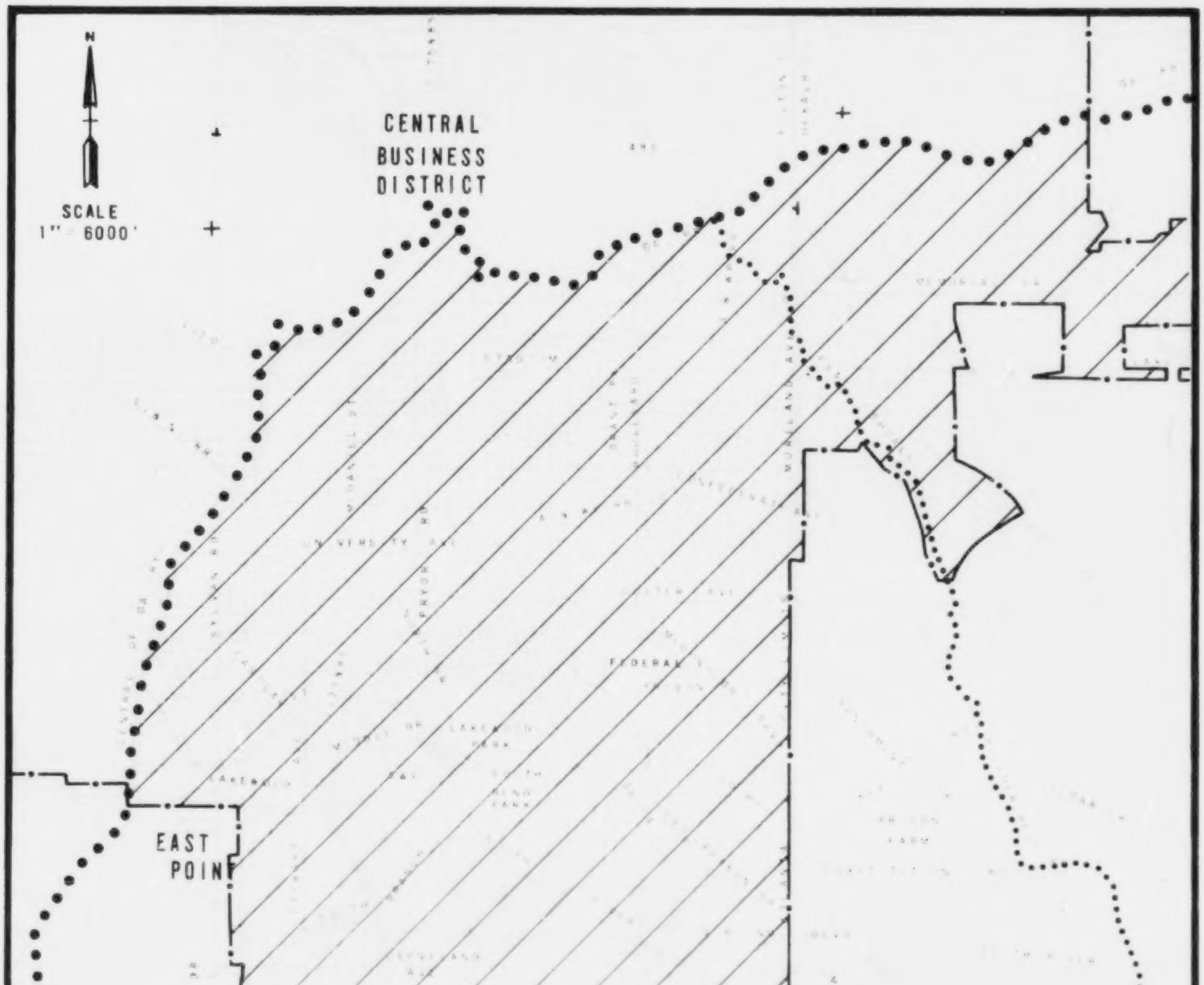

N
SCALE
1" = 6000'
CENTRAL
BUSINESS
DISTRICT
EAST
POINT
HAPEVILLE
ATLANTA
MOUNTAIN
VIEW
FOREST PARK
ATLANTA CITY LIMITS
STUDY AREA BOUNDARY
STORM AND COMBINED
SEWERS FIELD LOCATED
AND PLOTTED ON MAPS
SOUTH RIVER BASIN BOUNDARY
UPPER SOUTH RIVER
DRAINAGE BASIN
STUDY AREA
STORM AND COMBINED
SEWER MAPPING

Figure 2

Most of the upper South River basin is highly developed, but conditions vary widely with location, the northern portions being in general more developed than those to the south. Land uses vary accordingly, extremes ranging from dense downtown commercial and heavy industrial areas, to agricultural and entirely undeveloped tracts. Residential neighborhoods cover large fractions of the area. Industrial zones are generally concentrated on strips along the ridges, following location of railroad tracks.

Sewered Areas

Combined sewers are found in areas at the north end of the basin, where growth of the City was initiated during the last century. As required by urban growth and by the advent of plumbing and sanitary facilities within the City, natural drainage courses were converted, step by step, into combined sewers. These sewers were extended by sections, farther downstream, as the City grew. A variety of cross sections is found in these old combined sewers, many of which were built of dry masonry and stone slabs, or of brick and other construction materials.

When, early this century, deterioration of streams in urban areas became a problem because of increasing discharges of untreated sewage, construction of diversion works or regulator structures to separate dry weather flows, and of interceptors, to convey them to treatment plants, was initiated. These are the structures presently existing at combined sewer areas.

Other areas within the basin, developed later, are served by separate sewer systems of more recent construction.

More distant sections where development is only beginning to take place are still unsewered.

Areas of the City of Atlanta that are presently served by combined sewer systems are indicated in Figure 3.

South River Headwaters

At the headwaters, three separate branches join to form the South River, as can also be seen in Figure 3. The north branch presently originates as an open course at the McDaniel Street combined sewer overflow, its drainage area actually encompassing a combined sewer system with headwaters reaching the Spellman College campus. During dry weather, there is no flow in the first section of the north branch. Below the south expressway, a short distance downstream, it receives additional contributions from a smaller combined sewer serving the area adjacent to Pryor Road, north and south of University Avenue, and down to Joyland Place.

The branch flows southeast through the Lakewood Park area, bypassing the lake at the park's oval racetrack. Thence it flows under Lakewood Avenue and through South Bend

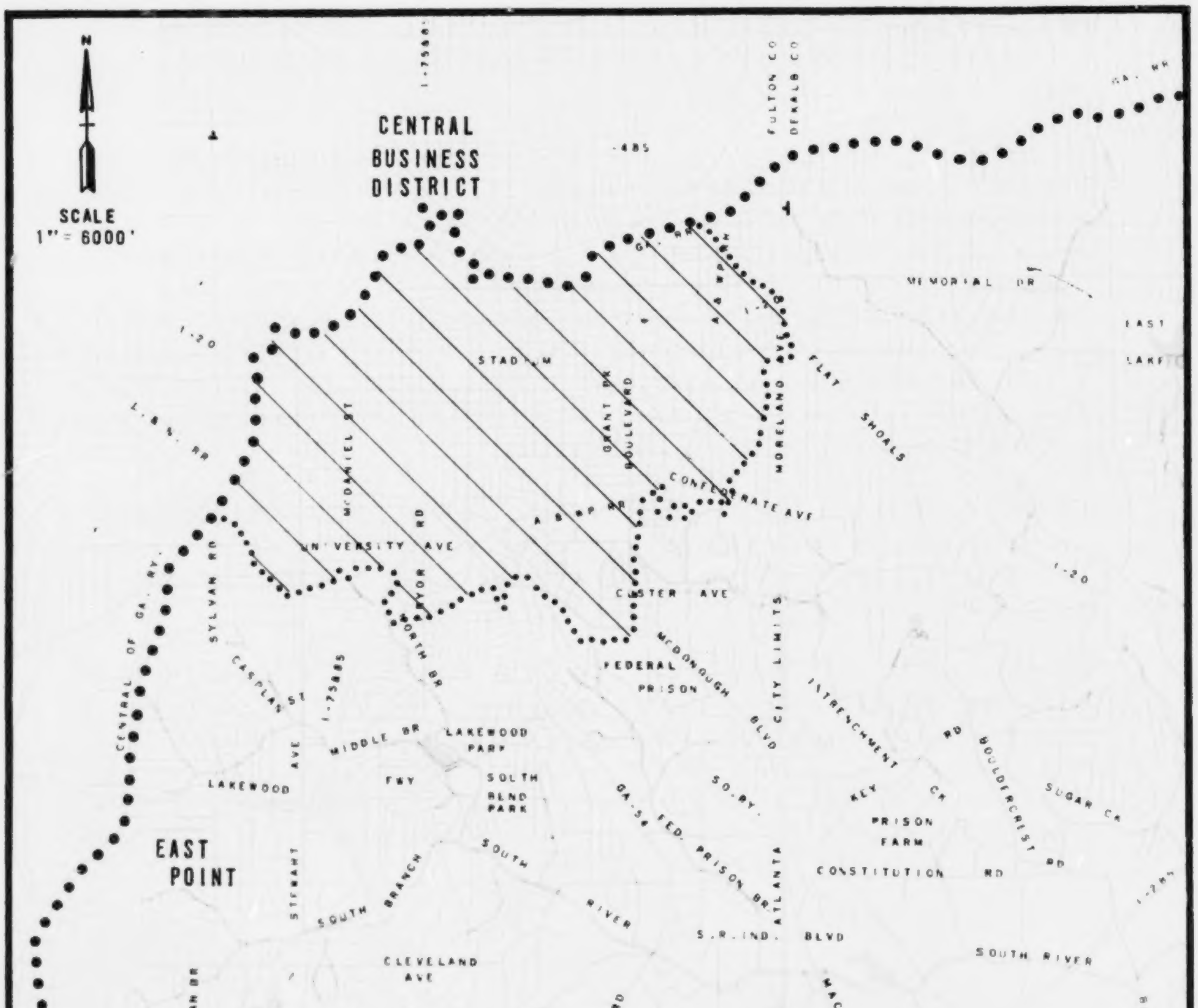

N
SCALE
1" = 6000'
CENTRAL
BUSINESS
DISTRICT
I-75S85
I-485
FULTON CO
DEKALB CO
GA. RR
MEMORIAL DR
EAST
LAKE
I-20
L & N. RR
STADIUM
MCDANIEL ST
GRANT PK
BOULEVARD
MORELAND AVE
FLAT SHOALS
CONFEDERATE AVE
A B & P RR
UNIVERSITY AVE
CUSTER AVE
CITY LIMITS
FEDERAL
PRISON
MCDONOUGH BLVD
SYLVAN RD
CENTRAL OF GA. RY.
CASPLAN ST
I-75S85
NORTH BR
MIDDLE BR
LAKEWOOD
PARK
LAKEWOOD
FWY
SOUTH
BEND
PARK
TRENCHMENT CK
KEY RD
BOULDERCREST RD
SUGAR CK
PRISON
FARM
SO. RY.
EAST
POINT
STEWART AVE
GA-54 FED. PRISON BR
ATLANTA
CONSTITUTION RD
SOUTH BRANCH
SOUTH RIVER
S.R. IND. BLVD
SOUTH RIVER
I-285
CLEVELAND
AVE
HARLAN RD
I-85
MACON DR
JONESBORO RD
CREEK
BOULDERCREST RD
HAPEVILLE
CITY LIMITS
POOLE
I-285
FULTON CO
DEKALB CO
C. OF GA. RR
ATLANTA
CITY LIMITS
I-85
ATLANTA AIRPORT
MOUNTAIN
VIEW
CLAYTON CO
GA-54
CONLEY
SOUTHERN RY
I-75
I-285
FOREST PARK
ARMY DEPOT
AREA SERVED BY
COMBINED SEWERS
SOUTH RIVER BASIN BOUNDARY
C. OF GA. RY.
SO. RY.
UPPER SOUTH RIVER
DRAINAGE BASIN
AREA WITH
COMBINED SEWERS

Figure 3

Park, where slopes are very steep, accounting for a fall of 50 feet in a distance of only about a thousand feet. Not far downstream it meets the other branches to constitute the South River.

The middle branch is formed at the confluence of two smaller streams draining the Sylvan Hills area, just west of Stewart Avenue and north of the Lakewood Freeway. One of these streams originates at the end of a storm sewer discharging at the south side of Deckner Avenue. It flows through Perkerson Park and under Casplan Street. A short distance downstream the two streams merge into the middle branch. The branch crosses under Stewart Avenue, continues through a number of structures that form a partially built storm sewer system, then flows through the Atlanta fair grounds and into the lake at Lakewood Park. The lake serves a flow regulation purpose. It overflows through a box culvert located at its southwest end, its waters reaching the north branch at a point about a quarter–mile below Lakewood Avenue in the South Bend Park area.

The south branch originates at the City of East Point and flows east into corporate Atlanta. It drains an area generally located south of the Lakewood Freeway and west of Interstate 75, and presently flows as an open stream for a distance of about four miles before reaching the confluence of the branches.

The north and south branches merge to form the South River a short distance south of South Bend Park, about 500 feet east of Macon Drive. From this point the river flows southeast to the Jonesboro Road bridge where a control station was established, and then almost due east until it leaves the City of Atlanta and enters DeKalb County. Just downstream of the Jonesboro Road bridge the river receives the effluent and occasional bypasses from the City's South River plant. Effluent volumes usually exceed base flows of the stream at this point.

Tributaries

The stream receives several tributaries within the upper basin, most of them from the north side. Most important for this study is Intrenchment Creek, originating at the overflow of the Boulevard combined sewer and also receiving overflows from the Confederate Avenue or Stockade combined sewer at a point near Georgia National Guard. The Intrenchment Creek basin encompasses part of downtown Atlanta, where the State Capitol and City Hall are located. Intrenchment Creek flows east until it crosses Atlanta city limits, then runs approximately southeast through DeKalb County to the location of Atlanta's Intrenchment Creek Wastewater Treatment Plant. Effluent from this plant, and occasionally bypassed flows, are discharged into the creek. Intrenchment Creek enters the South River 1.1 miles upstream of Bouldercrest Road bridge.

Downstream of Bouldercrest the South River basin widens. By the time the stream reaches the Klondike Road bridge, 16 miles below, the drainage area encompasses approximately 182 square miles and includes small fractions of Henry and Rockdale counties. A large number of tributaries join the river upstream of Klondike. Most important of them, from the north side, are Sugar Creek, Doolittle and Doless Creeks, Shoal Creek, Cobbs Creek, Snapfinger and Barbashela Creeks and Pole Bridge Creek. Poole Creek, Blue Creek, and Conley Creek enter the river from the south. Many of these tributaries receive polluted storm runoff from urban areas and discharges from municipal and industrial outfalls. The basins of Sugar, Doolittle and Doless, and Shoal Creeks encompass highly developed areas.

Pollution Sources

Known pollution sources within the area are numerous, and a complete analysis and evaluation of all of them is almost impossible within reasonable limits of time and cost. Pollutional contributions to the stream from combined sewer overflows, storm runoff from certain areas, treatment plant effluents, and bypassed flows are considered most significant and assessment of their magnitudes has been undertaken. Highly significant also, but more difficult to estimate, is pollution from industrial wastes, continuously or occasionally reaching the stream. Numerous potential industrial sources of pollution are found to exist within the basin. A complete investigation of the industrial waste problem, however, exceeds the study limitations.

SECTION III

METHODS OF STUDY

Location and Mapping

Storm and Combined Sewer Systems

Since maps of the storm and combined sewer systems were not available, the task of location, leveling and mapping of these systems was included in the project and scheduled for completion at an early date.

Surveying crews established a network of reference elevations, located in the field and plotted on aerial contour maps all existing elements of these systems, and obtained invert and top–of–rim elevations at all manholes, catchbasins and pipes at headwalls or other hydraulic structures. Sewer sizes and materials were also investigated and included in the mapping for the systems. Maps are on matching reproducible cronaflex to a scale of one inch equals 200 feet and have 2–foot contours. The set consists of fifty sheets, including an index. They served as a basis for the study.

Assessment of the Infiltration Problem

As a part of the storm and combined sewer survey, an assessment of the infiltration problem was made. Visual inspection at every manhole of the combined sewer system indicated that infiltration is not a significant problem in this area. This conclusion was supported by information obtained through the City of Atlanta from their sewer maintenance crews. Therefore it was deemed unnecessary to conduct additional conductivity measurements, flow observations and smoke tests to locate leaks and illicit connections. An abundance of clays and other impervious materials contributes to this lack of an infiltration problem.

Establishment of Basin Boundaries

Boundaries for the upper South River basin, and for each of the smaller drainage areas given separate consideration in the study, have been established from the contour and storm drainage maps described before. Contours at 2–foot intervals and detail of the existing sewer systems permit accurate determination of drainage boundaries. For areas outside corporate Atlanta, U.S. Geological Survey quadrangles and maps from Atlanta Region Metropolitan Planning Commission and other sources were utilized.

Selection of Monitoring Sites

As a part of the statement of work included in the contract, monitoring sites within the study area were selected during the preliminary steps of the study. Selection included all

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three known major overflow points, as well as interceptors originating at these points. It also included three points at streams fed from storm runoff, for purposes of comparison. Control stations along the river were located upstream of the outfall of South River wastewater treatment plant and also at points below, where pollutional loads from this and the Intrenchment Creek plant effluents and bypasses could be expected to influence more directly conditions in the stream. A bypass at the interceptor entering Intrenchment Creek plant was also monitored. Finally, a station was established at the Klondike Road bridge, located about 20 miles downstream of the South River plant, where the river shows recovery at normal or dry weather flow conditions. Location of all monitoring sites and of recording rain gages appears in Figure 4.

Recording Gage Network

A first–order Environmental Science Services Administration (ESSA) station exists at Atlanta airport. However, large variations in distribution, timing, and other storm characteristics were expected between the small drainage areas under study. It was anticipated that a rather dense recording rain gage network would be necessary.

Four recording rain gages were installed early during the study at selected locations within the upper South River basin. Information from these stations was to be supplemented by that from the Airport station and from an existing recording gage maintained at the Atlanta Stadium, in the center of the basin's combined sewer area.

The four project rain gages were of the weighing and recording type, Catalog No. 5–780 from Belfort Instrument Company, Baltimore, Maryland. They consist of four major elements: The collector, the weighing mechanism, the chart drive, and the housing. A photograph of one of the installed instruments and a reproduction of a typical record chart can be seen in Figure 5. The collector diameter is eight inches. Weighed amounts of precipitation are recorded on a chart as inches of rainfall, and can be read to the nearest hundredth of an inch. Charts are six inches high; recorders are single traverse type. The chart drive is spring wound and runs eight days in one winding. All instruments were equipped with time scale gears adequate for operation with eight–day charts. Later in the study, gears were replaced in gages at Stations 51 and 52 to operate on one–day charts, permitting a much more accurate reading of rainfall in short time intervals.

Establishment of Gaging and Sampling Stations

Measurement of quantity and quality of flows entering the stream, and flowing at different points along it, was one of the key aspects of the investigation.

The nature of conditions at these monitoring sites necessitated both continuous and event monitoring.

Continuous monitoring was considered to be required at stations along the river and at smaller streams where a base flow was found to exist at all times. It was also established at the interceptors where dry weather flow characteristics were of interest to the investigation.

Event monitoring was needed mainly at overflow points where no flow exists at normal dry weather conditions, and at bypass lines to existing treatment facilities.

Flow Measurement

Flow monitoring was generally accomplished through installation of flow level recorders and development of rating curves at each of the selected monitoring sites.

Flow level recorders utilized were Stevens, Type F, Leupold & Stevens Instruments, Inc., Portland, Oregon. Construction of gaging stations and general installation details were in accordance with U.S. Geological Survey established practice. Stilling wells were fabricated of 14–gage corrugated aluminum pipe, with bottoms and cleanout doors. Water inlet pipes were sized to dampen surges of the water surface as required by stilling well diameters.

For flow level recording at interceptors, scow floats were installed at selected manholes a short distance downstream of the regulators.

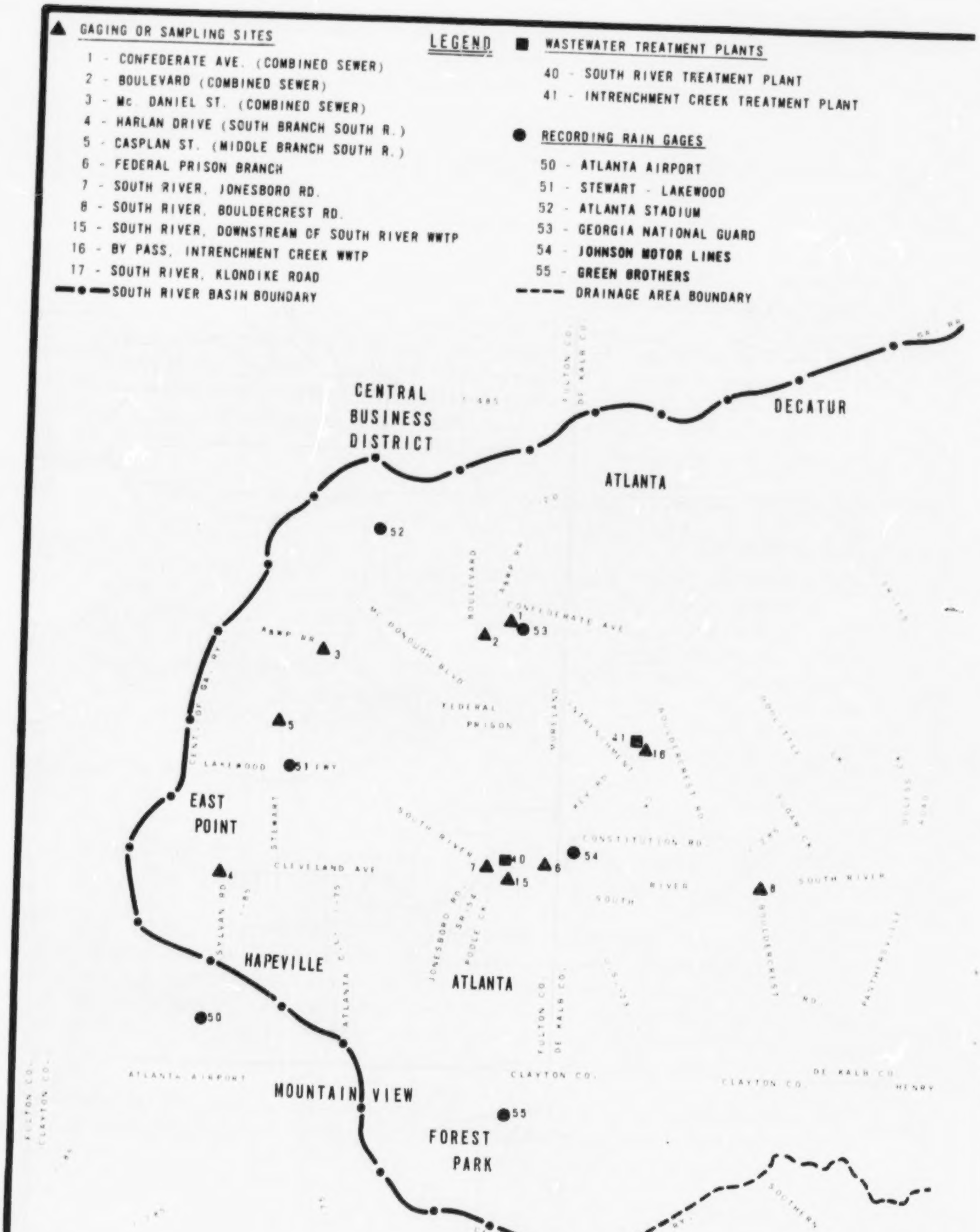
Rating curves for all gaging stations were developed by stage–discharge measurements with current meters. Price Type AA current meters, Scientific Instrument Co., Inc., Milwaukee, Wisconsin, were employed for discharge measurements within the velocity and depth ranges of these instruments. For low flow and shallow depth measurements, Pigmy type current meters mounted on top setting wading rods were used. Whenever possible, discharge measurements were verified by use of alternate methods or formulas in addition to current meter measurements at certain stages. Samples of discharge measurement notes and rating curves are included in Appendix D.

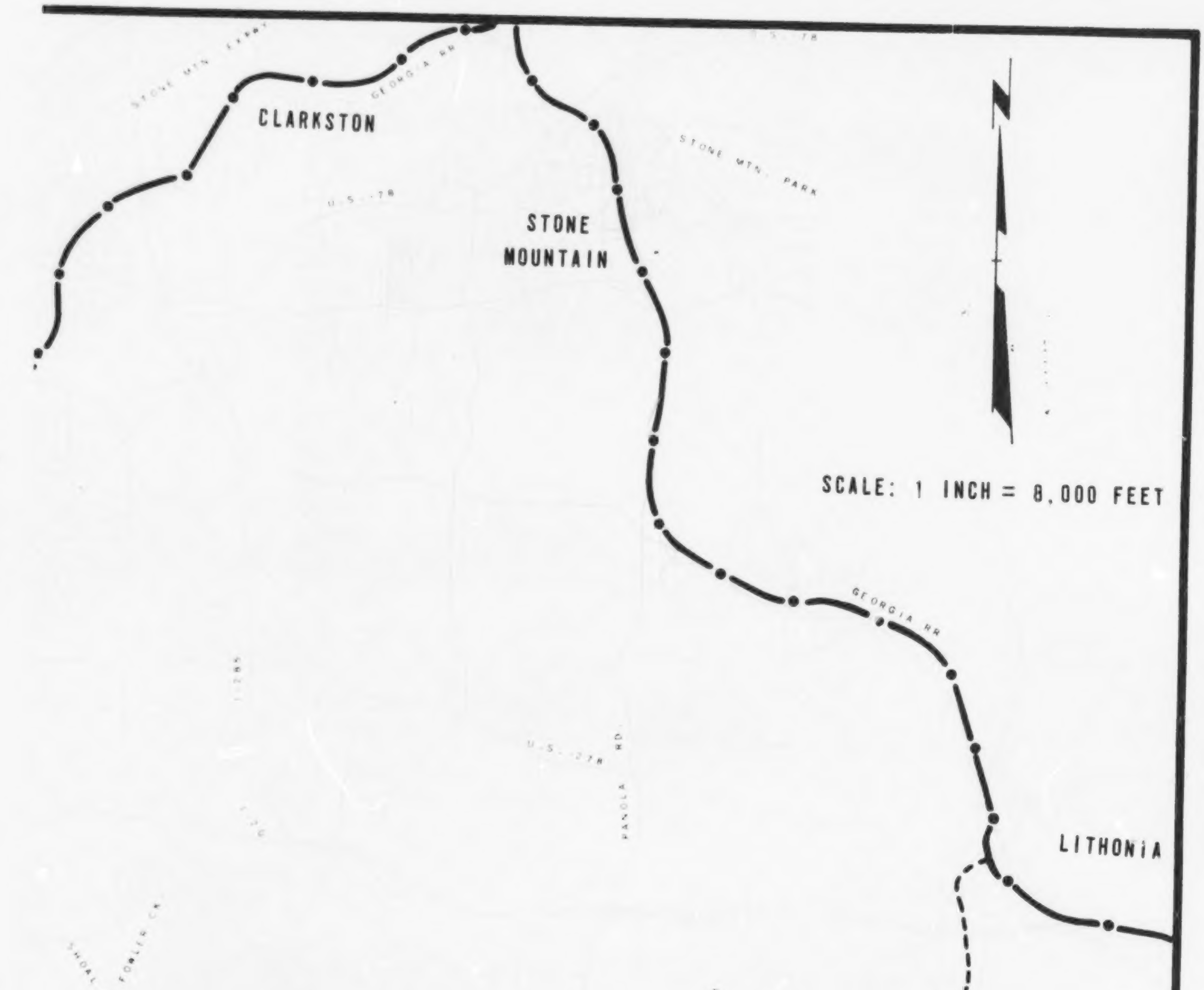
Sampling Procedures

Both grab and automatic sampling procedures were employed during the investigation. Automatic sampling was more extensively used, generally being more advantageous.

Four Serco Model NW 3–8 automatic samplers, Sanford Products Corporation, Minneapolis, Minnesota, were utilized. These samplers operate on a preset vacuum, and are capable of collecting 24 sequential samples at equally spaced time intervals. Depending upon the clock being used, five–, fifteen–, and sixty–minute sampling intervals can be obtained. Samples are collected through individual flexible hoses ending at a protective sampling head that is placed at the stream or sewer to be sampled.

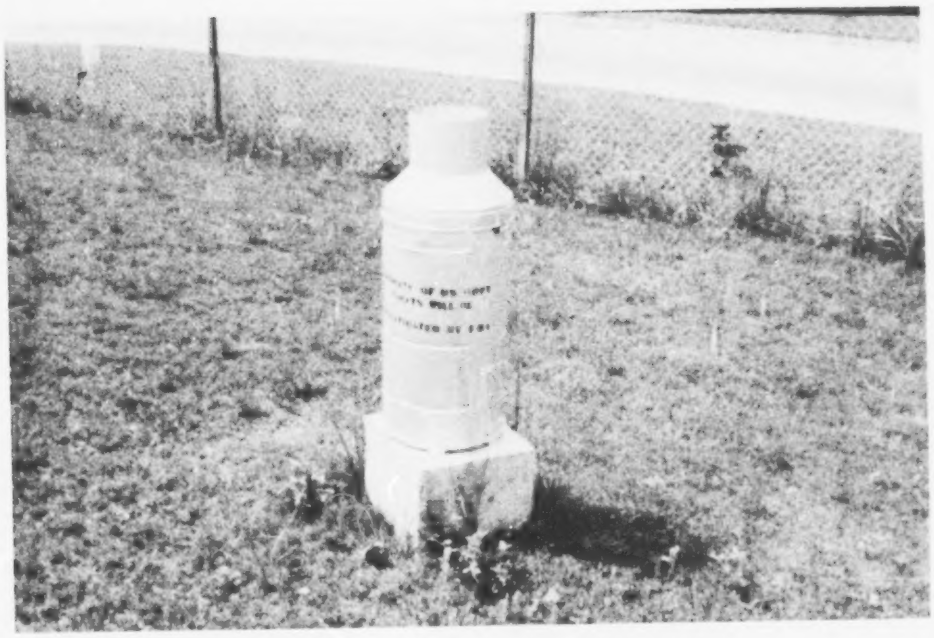
Automatic sampling at points of continuous flow was accomplished by simply starting clock mechanisms of the samplers at the selected times. Application of vacuum to the sample


▲ GAGING OR SAMPLING SITES
1 - CONFEDERATE AVE. (COMBINED SEWER)
2 - BOULEVARD (COMBINED SEWER)
3 - Mc. DANIEL ST. (COMBINED SEWER)
4 - HARLAN DRIVE (SOUTH BRANCH SOUTH R.)
5 - CASPLAN ST. (MIDDLE BRANCH SOUTH R.)
6 - FEDERAL PRISON BRANCH
7 - SOUTH RIVER, JONESBORO RD.
8 - SOUTH RIVER, BOULDERCREST RD.
15 - SOUTH RIVER, DOWNSTREAM OF SOUTH RIVER WWTP
16 - BY PASS, INTRENCHMENT CREEK WWTP
17 - SOUTH RIVER, KLONDIKE ROAD
SOUTH RIVER BASIN BOUNDARY
LEGEND
■ WASTEWATER TREATMENT PLANTS
40 - SOUTH RIVER TREATMENT PLANT
41 - INTRENCHMENT CREEK TREATMENT PLANT
● RECORDING RAIN GAGES
50 - ATLANTA AIRPORT
51 - STEWART - LAKEWOOD
52 - ATLANTA STADIUM
53 - GEORGIA NATIONAL GUARD
54 - JOHNSON MOTOR LINES
55 - GREEN BROTHERS
DRAINAGE AREA BOUNDARY
CENTRAL BUSINESS DISTRICT
I-485
FULTON CO.
DE KALB CO.
DECATUR
GA. RR.
ATLANTA
I-20
52
BOULEVARD
A&WP RR
CONFEDERATE AVE.
1
2
53
A&WP RR
3
Mc DONOUGH BLVD.
CENT. OF GA. RY.
FEDERAL PRISON
MORELAND
INTRENCHMENT CR.
BOULDERCREST RD.
41
16
5
LAKEWOOD
51 FWY
EAST POINT
STEWART
SOUTH RIVER
CONSTITUTION RD.
7
40
6
54
15
4
CLEVELAND AVE.
SYLVAN RD.
I-85
I-75
SOUTH RIVER
8
SOUTH RIVER
BOULDERCREST RD.
PANTHERSVILLE
JONESBORO RD.
SR 54
POOLE CR.
HAPEVILLE
ATLANTA
FULTON CO.
DE KALB CO.
50
ATLANTA AIRPORT
FULTON CO.
CLAYTON CO.
CLAYTON CO.
DE KALB CO.
CLAYTON CO.
HENRY
MOUNTAIN VIEW
55
FOREST PARK
SO. RY.
SOUTHERN RY.
CENT. OF GA. RY.

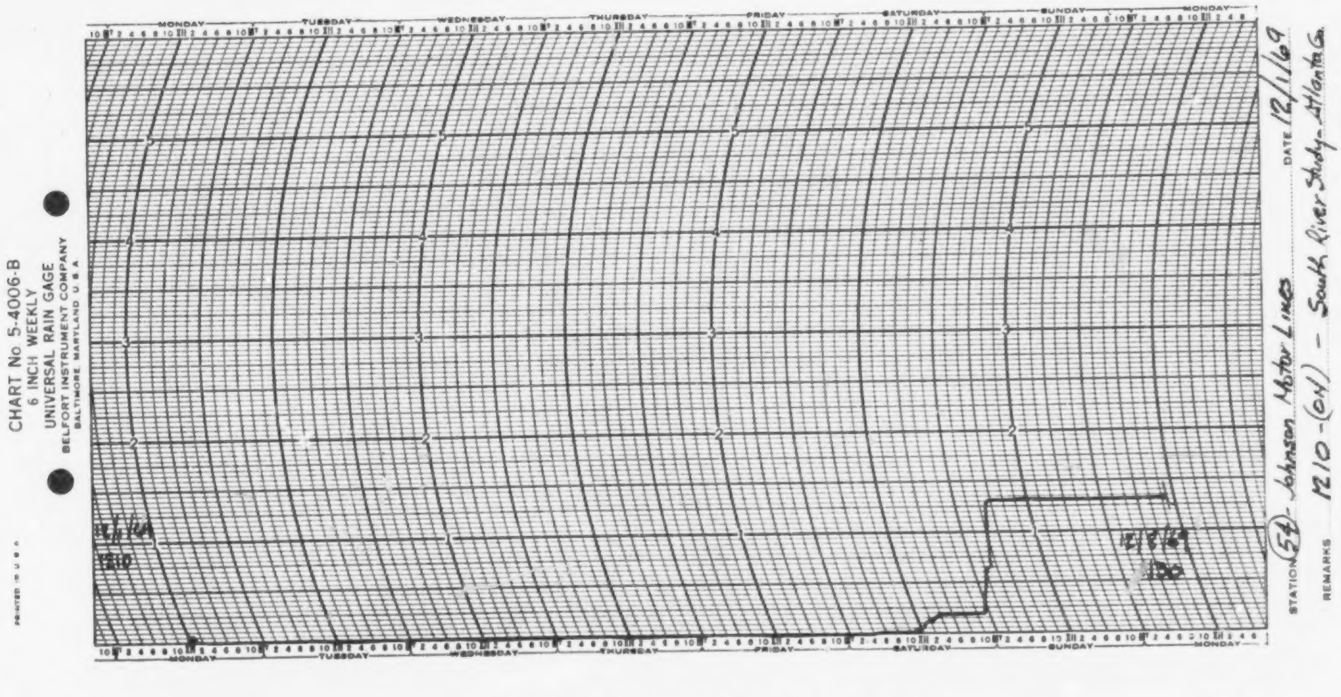

STONE MTN. EXPWY
GEORGIA RR
U.S. 78
CLARKSTON
STONE MTN. PARK
U.S. 78
STONE
MOUNTAIN
N
SCALE: 1 INCH = 8,000 FEET
GEORGIA RR
I-285
U.S. 278
PANOLA RD.
I-20
LITHONIA
SHOAL CK.
FOWLER CK.
SR-155
POLE BRIDGE CREEK
SNAPFINGER CK.
SOUTH RIVER
SR-155
DE KALB CO.
HENRY CO.
SR-212
KLONDIKE ROAD
SOUTH
RIVER
17
DE KALB CO.
ROCKDALE CO.
ROCKDALE CO.
CLAYTON CO.
HENRY CO.

LOCATION OF STATIONS WITHIN STUDY AREA

Figure 4



Recording Raingage
Stewart Avenue North of Lakewood Freeway
(Station 51)



Typical Raingage Weekly Chart

Figure 5

bottles was performed with a vacuum pump in the laboratory. At each sampling station an adequate setup for the samplers was prepared, including means for protecting them from theft or vandalism.

Samples were retrieved at the end of each sampling period, usually within four to six hours, and were taken to the laboratory for analysis. A significant fraction was collected during the cool weather part of the year, offsetting the need for sample refrigeration prior to retrieval and analysis.

For event sampling, as in the case of overflows and bypasses, an automatic triggering device was developed so that automatic sampling would be started only when flow was initiated at the sampling station. The purpose of this type of sampling was primarily to monitor events beginning at times when no personnel would be available to manually start the sampler or to take grab samples.

The triggering devices prepared for use in conjunction with the automatic samplers consisted of a solenoid capable of unblocking the escape of the sampler's clock when energized by means of a battery operated electric circuit. Closing of the circuit was accomplished by contact of a stainless steel wire with a screw in the wheel of the flow level recorder. Contact adjustment was easy, permitting start of the clock at a predetermined flow depth increase. Four hundredths of a foot was generally selected. With this arrangement, the first sample was taken one sampling interval after beginning of the event.

Operation of triggering devices was successful when first used, but many difficulties were soon experienced. Although high voltage batteries were utilized, electrical contacts proved deficient, and the apparently well–insulated solenoids were affected by dampness. Consequently during the latter part of the study, the sampler had to be triggered by hand. It was also found that equal time–interval sampling was not desirable for some events, and that too much time was elapsing between the beginning of the event and collection of the first sample.

Main difficulties with grab sampling of the initial phases of overflows were with personnel availability at the right time and place. Commencement of rainfall proved difficult to predict. After storms commenced, traffic conditions always worsened in the area, making it impossible to reach the sampling locations in time.

When possible, grab sampling of the initial overflow discharges gave a better coverage of the variation of quality with time during these events. Simultaneous grab and automatic sampling at a few stations for verification of the performance of automatic samplers in obtaining representative samples produced no significant differences.

Difficulties were encountered in establishment of rating curves, due to the short time allocated for data acquisition. The large number of sites to be gaged, the unpredictability of heavy storms and the variability of available personnel combined to limit the number of occasions during which rating curve points might be established at each site. The unsteady peak rates of storm runoff hindered accurate gaging at these high flows.

Until sufficiently well defined rating curves were developed, it was not possible to relate concentrations of water quality parameters to flow rate, as discussed subsequently in this report. This in turn delayed assessment of areas for which additional water quality data were needed to improve statistical validity of conclusions. For this reason the six months originally scheduled for data acquisition proved to be insufficient.

Additional problems were encountered with location of stilling wells and weirs at sites yielding very high flow velocities during periods of runoff. One complete stage–level monitoring station and one weir were lost during the period of study. Equipment was generally located in partially protected areas with lower flow velocities. Any error due to this adaptation of the ideal procedure should be minimal since the rating curves related flow to water level in the stilling well.

Measurement of storm flows in the interceptors occasionally was complicated by surcharging within the manhole, especially at the McDaniel Street interceptor. This caused the scow float to rise to the top of the pipe, preventing accurate flow measurement.

Sample volumes obtained by the automatic samplers were insufficient to perform the full range of tests conducted during the study, thereby necessitating partial analyses on each sample. Due to large stage variation during overflows, samplers had to be located high above low flow levels. Vacuum was therefore insufficient to fill the sampling bottle, since considerable portions remained in the sampling tube. Collection of larger samples would have required a much larger and less maneuverable device. Furthermore, during the winter months, problems were encountered with samples freezing in the long tubes before reaching the bottles.

Laboratory Establishment – Sample Analysis

To facilitate transportation of samples for analysis and reduce elapsed time between sampling and testing, a laboratory within the study area was established. The laboratory was located, through cooperation of the City of Atlanta, in the Intrenchment Creek wastewater treatment plant, and was equipped to perform all analyses required by the contract. Capacity of the laboratory was based upon handling batches of 96 samples at peak load times.

All testing was done in accordance with Standard Methods for the Examination of Water and Wastewater, 12th Edition, 1965. To facilitate laboratory work, a brochure was prepared with simplified, step–by–step descriptions of all procedures for the physical and chemical examinations to be performed. The laboratory was organized to test for biochemical oxygen demand (BOD), chemical oxygen demand (COD), dissolved oxygen (DO), total suspended solids, volatile suspended solids, settleable solids, conductivity, pH, acidity, alkalinity, total and orthophosphates, chlorine residual, total residue, and volatile residue. A reasonable number of tests were conducted on each sample. Bacteriological determinations included total coliform, fecal coliform, and fecal streptococci.

Continuous Quality Monitoring

Conductivity, temperature, and disolved oxygen were continuously monitored in the field by means of portable, battery–powered recording equipment. Conductivity recorders were Beckman Model RQ–1, Beckman Instruments, Inc., Fullerton, California, with automatic temperature compensation. Charts and clock mechanisms for weekly and 24–hour recording were used. Some problems were encountered with the recording mechanisms of these recorders.

For dissolved oxygen and temperature monitoring, Rustrak Model 192 recorders, Rustrak Instrument Division, Gulton Industries, Inc., were used. Difficulties were found with operation of oxygen probes when flow velocities were low at locations being monitored. Once clogged at low flow velocities, the probes failed to clear with an increase in flow. The recording charts permitted continuous monitoring for approximately one–month periods, however, the probes generally failed to operate after a very limited amount of time. Personnel limitations precluded suffuciently frequent attention to these remote recorders.

Data Collecting and Processing

Data from the stage level recorders and laboratory analyses were reduced to tabular form and punched onto IBM computer cards for subsequent analysis on the University of Florida IBM 360/65 computer. A computer program was written to interpret all overflow events at each combined sewer station and present them as outflow hydrographs. Cumulative flows for each event were also calculated. A similar program was used to present dry and wet weather flows at other monitoring sites. Samples of the printouts for a few events are included in Appendix D. Hydrograph similarities between two nearby overflow points can be noticed during a winter or frontal type storm.

Another program tabulated all chemical analyses for each time increment during overflow events, and calculated the pollution loading rates for each parameter in 1,000 pounds per day. This procedure enabled periodic analysis of rainfall, runoff, and water quality data in order to select overflow events suitable for further detailed study. A sample of chemical analysis results on a computer printout sheet is included in Appendix D.

SECTION IV

RAINFALL AND RUNOFF

Existing Local Precipitation Records

Good climatological data records are available for the Atlanta area in general. An ESSA (Environmental Science Services Administration) station is located at the Atlanta airport, outside the South River basin, but only two–thirds of a mile west of the basin divide. Hourly precipitation records are available for this station, and were utilized to supplement data obtained from the rain gages installed at selected points within the study area.

The Atlanta station was established in 1878 at the Kimball House in downtown Atlanta. It occupied this and several nearby locations until 1954, when the last instruments in use were removed. The Airport station was established in 1928 and its records became official for the Atlanta area beginning December 1, 1934.

Mean annual rainfall for the 1879–1969 period of record is 48.46 inches. This figure is not adjusted for instrument location changes that have occurred during the 90 years. For the 39–year period beginning in 1931, the maximum annual precipitation was 71.45 inches in 1948, and the minimum 31.80 inches in 1954. Long range records indicate that the minimum monthly mean precipitation occurs in October and the maximum mean in March.

Storms in the area are basically of two different types: winter storms of the frontal type, generally of long duration resulting in moderate to heavy precipitation; and summer thunderstorms, generally brief but more intense.

Rainfall Intensity–Duration–Frequency Curves

Rainfall intensity–duration–frequency curves for the area have been developed by the U.S. Weather Bureau using the method of extreme values, after Gumbel, for the period of record 1903 to 1951. These curves are reproduced in Figure 6. Means and extremes of monthly precipitation, as well as maximum 24–hour records for a 35–year period (1934–1969), are shown in Table 1. Monthly totals and greatest 24–hour figures for 1969 are also included in this table. For storms more frequent than two years it may be assumed that the extreme value distribution does not fit. A better fit would be obtained with a log–normal distribution. Such frequencies are of interest to combined sewer overflow problems. Intensity–duration–frequency curves for storms of less than a two–year frequency were therefore constructed.

Records from the Atlanta airport (ESSA) station were chosen, to enable subsequent refinement of the curves if this becomes necessary. The period from July, 1968, to February, 1970, comprising 20 months of hourly rainfall data, was analyzed. Rainfall during

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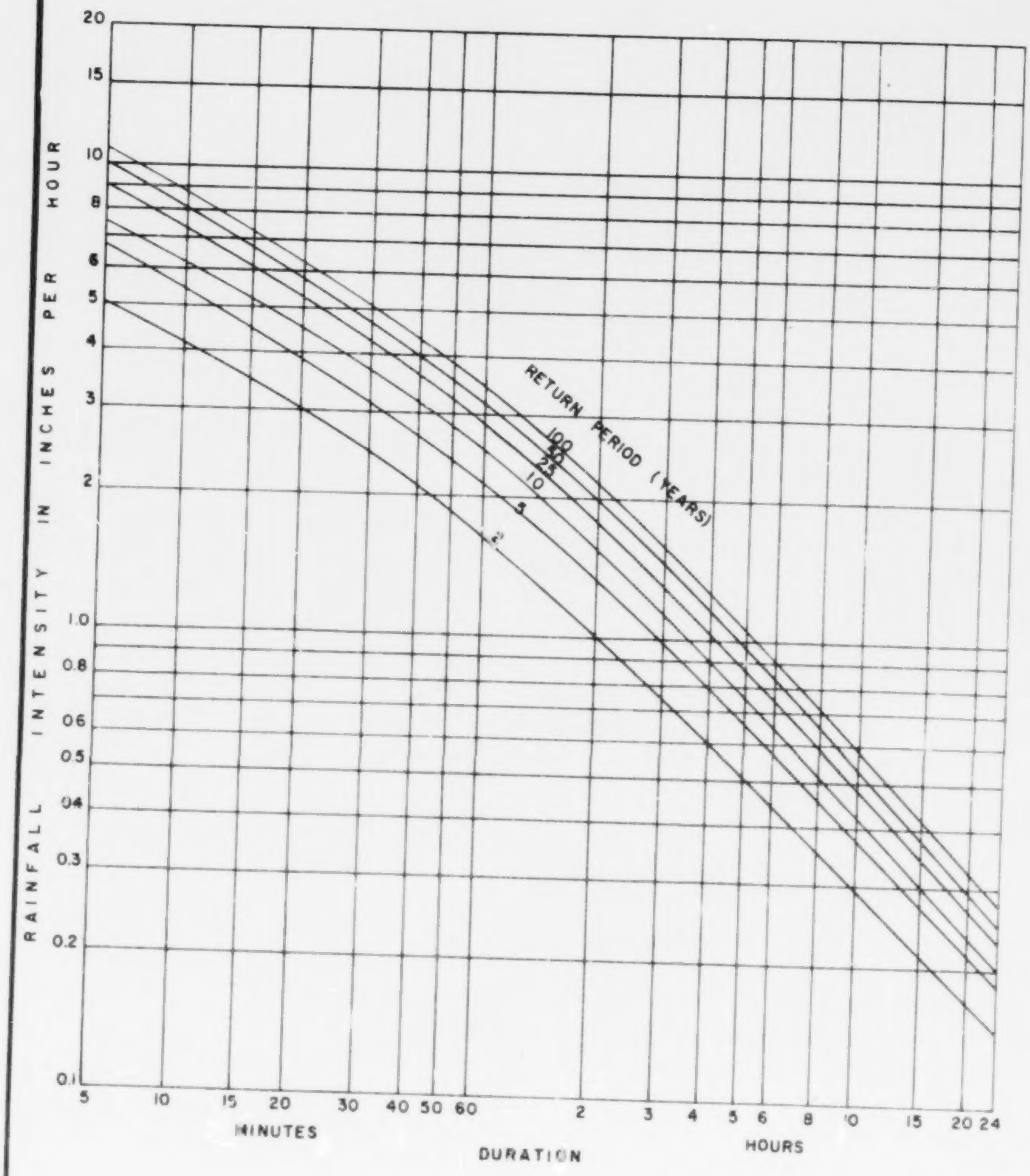


SOURCE: U.S. WEATHER BUREAU
TECHNICAL PAPER NO.25

PERIOD OF RECORD: 1903 - 1951
FREQUENCY ANALYSIS BY METHOD OF
EXTREME VALUES, AFTER GUMBEL

CITY OF ATLANTA, GEORGIA
RAINFALL INTENSITY-
DURATION-FREQUENCY CURVES

Figure 6

TABLE 1

PRECIPITATION IN INCHES
ATLANTA AREA(1)

		1935 – 1969 Period of Record			Calendar Year 1969	
Month	Normal Total(2)	Maximum Monthly	Minimum Monthly	Maximum 24 Hours(3)	Month Total	Greatest in 24 Hours
January	4.44	10.82	1.42	3.27	2.85	1.65
February	4.51	12.77	0.99	5.67	3.20	0.71
March	5.37	11.51	2.73	4.82	4.00	1.53
April	4.47	9.86	1.45	4.26	5.70	2.69
May	3.16	7.83	0.32	5.13	7.68	4.34
June	3.83	7.52	0.74	3.14	1.00	0.43
July	4.72	11.26	1.20	5.44	2.64	1.51
August	3.60	8.69	0.88	5.05	6.12	1.79
September	3.26	7.32	0.26	5.46	3.74	1.89
October	2.44	7.53	T	3.27	1.53	0.69
November	2.96	15.72	0.41	4.11	2.67	1.44
December	4.38	9.92	1.08	3.85	3.27	1.11
Year	**47.14**	**14.72**	**T**	**5.67**	**44.40**	**4.34**

(1) Atlanta Airport Station Records Official for Atlanta Area beginning December 1, 1934.

(2) Climatological Standard Normals (1931 – 1960).

(3) Maximum 24–hour precipitation on record: 7.36 inches, March 1886.

T = Traces

Source: U.S. Department of Commerce, Environmental Science Services Administration this period was 4.7 percent below normal, based upon 30 years of record (1931 to 1960). Rainfalls of less than 0.04 inch per hour were excluded, as was one five–year storm that occurred during the period. The resulting curves for two–week, one–month, six–month, and one–year storms are shown in Figure 7.

Recorded daily precipitation for the year 1969 is shown graphically in Figure 8 for the Atlanta airport station and for the study's rain gage network. Location of rain gages can be seen in figure 9.

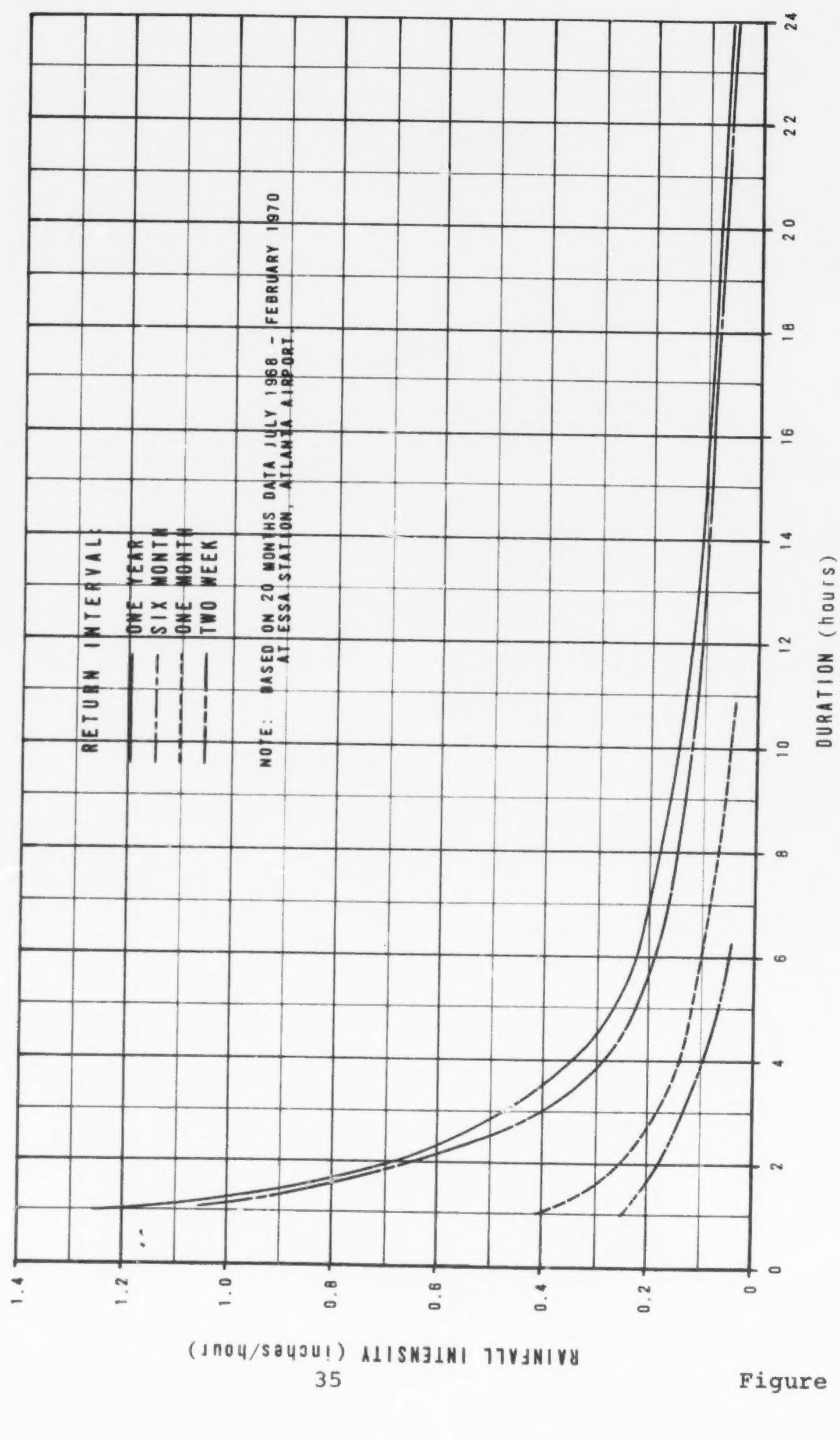
Availability of records from the Atlanta Stadium station (No. 52) was lost by the end of March, 1969. It was not, however, until some later date that this was realized, since this station was not maintained by project's personnel. To replace the stadium station, the project rain gage at Station 55 was removed and installed at the City's Fire House No. 5, adjacent to the stadium, this location being most important for the study's purpose. Records for the storms of April and May, 1969, lost at the stadium station, could not be obtained from a recording gage operating then at the Georgia Institute of Technology, since charts for these months were reportedly misplaced.

Daily recordings were also obtained for the Peachtree–DeKalb airport station located eleven miles north–northeast of the center of the rain gage network. Fifteen–minute recordings for selected storms are also available from this station. For the main purposes of this study, however, records from this station were of little help, due to significant differences in times of events. Differences in rainfall characteristics between this station and those in the project's rain gages for given storms confirm the need for a dense rain gage network, as well as short–interval rainfall intensity measurement, for studies of this kind. Accurate synchronization of all stations is also essential for adequate correlation of records.

Observed Rainfall Distribution in Study Area

Total precipitation during 1969 was calculated at each of the five ainfall stations in the study area and results were augmented by records from Atlanta and DeKalb–Peachtree airports, Robbins Air Force Base and a point near the Fulton County airport. The records range from a high of 54.30 inches at DeKalb–Peachtree airport eleven miles to the north–northeast, to a low of 44.40 inches at the Atlanta airport six miles to the south–southwest of the study area. Stations 51 and 52, which monitor rainfall for the three areas of greatest importance in this study, averaged 53.54 inches per year. The data are very limited; however, a possible trend may exist with rainfall decreasing in a southwesterly direction, reflecting the climatological influence of the nearby Blue Ridge mountains. The effect of rainfall distribution is discussed in Section V.

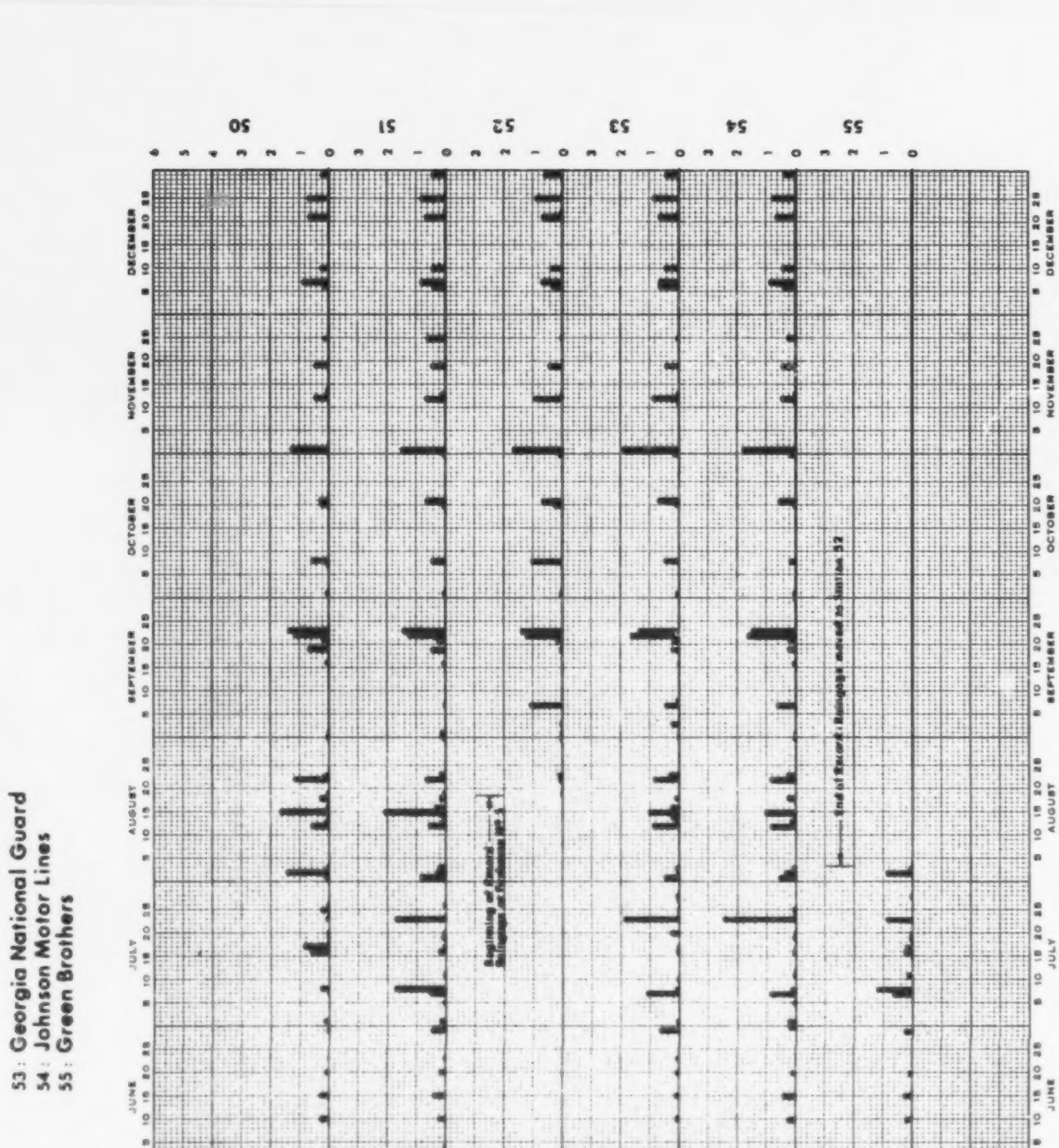
The 118 storms occurring during 1969, as recorded at the Atlanta airport, were categorized to determine their volume distribution. Fifty–six of these storms yielded 0.10 inch of rain or less, while the largest storm produced 4.34 inches. The distribution is plotted in Figure 10.



NOTE: BASED ON 20 MONTHS DATA JULY 1968 - FEBRUARY 1970 AT ESSA STATION, ATLANTA AIRPORT.

RAINFALL INTENSITY-DURATION-FREQUENCY CURVES, ATLANTA, GEORGIA

Figure 7



RECORDED TOTAL DAILY PRECIPITATION
CALENDAR YEAR 1969

Figure 8

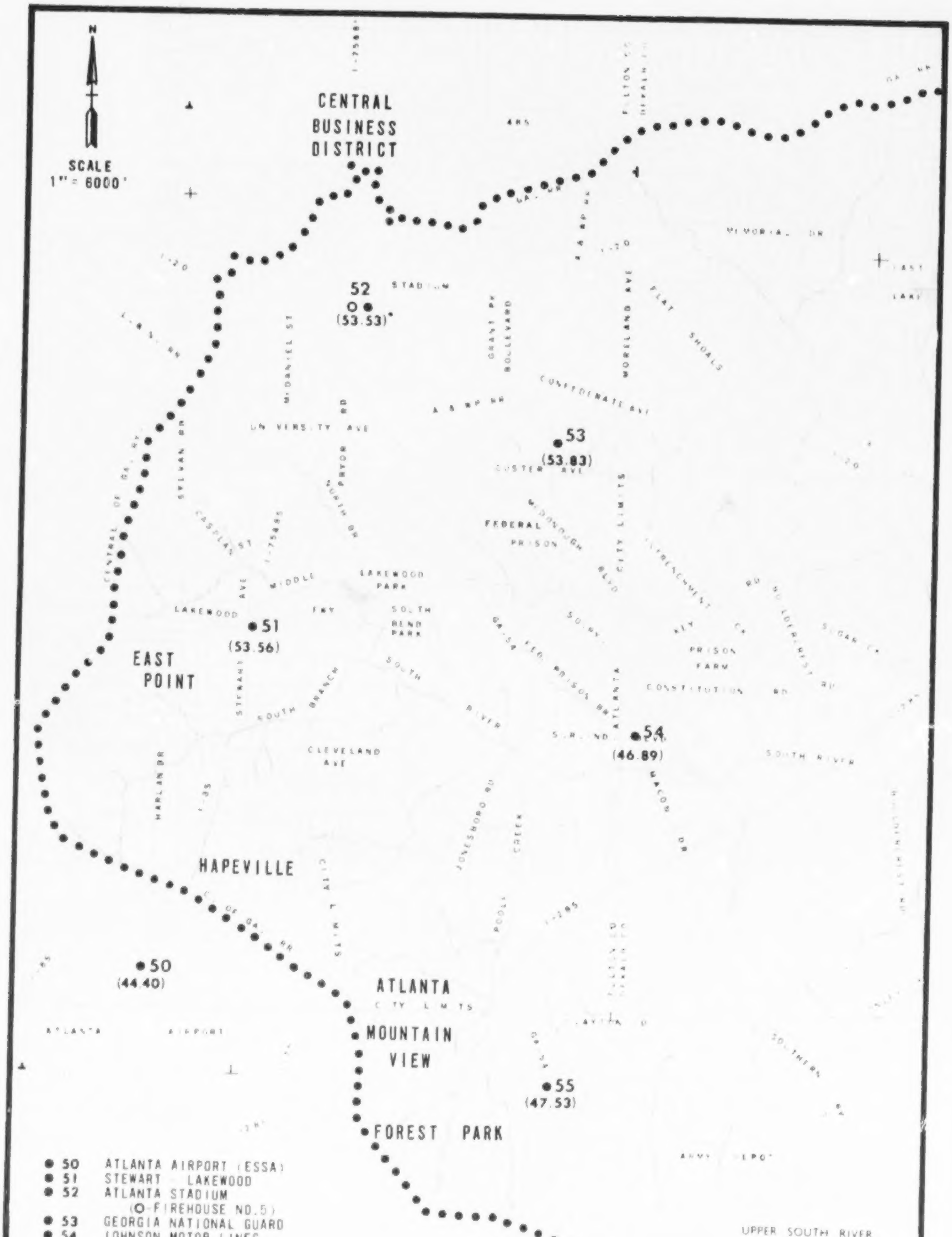
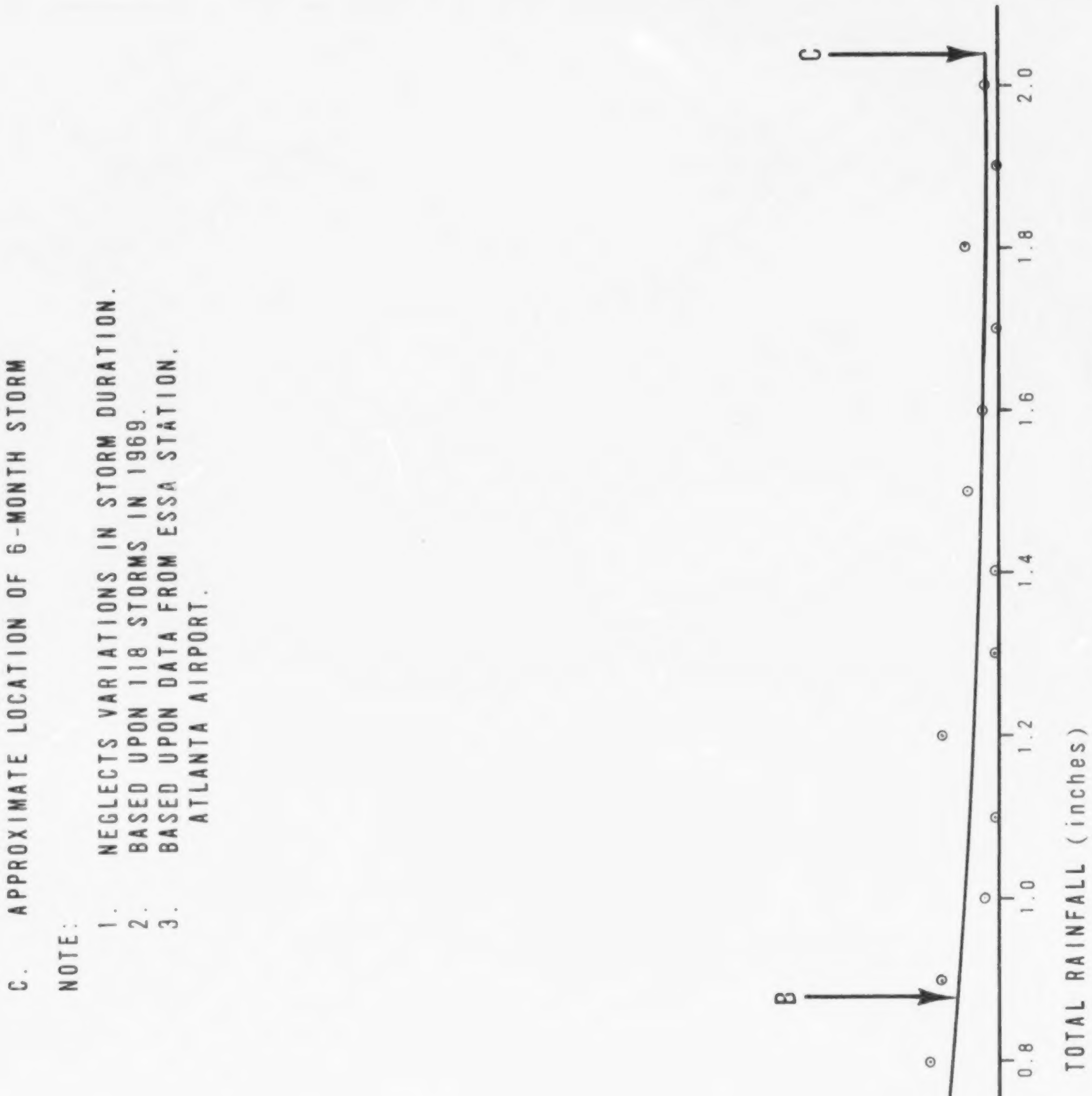

N
SCALE
1" = 6000'
CENTRAL
BUSINESS
DISTRICT
52
(53.53)*
STADIUM
53
(53.83)
51
(53.56)
54
(46.89)
50
(44.40)
55
(47.53)
EAST
POINT
HAPEVILLE
ATLANTA
MOUNTAIN
VIEW
FOREST PARK
FEDERAL
PRISON
LAKEWOOD
PARK
SOUTH
BEND
PARK
PRISON
FARM
ARMY DEPOT
ATLANTA AIRPORT
UNIVERSITY AVE
CUSTER AVE
MEMORIAL DR
CONFEDERATE AVE
MORELAND AVE
FLAT SHOALS
GRANT PK BOULEVARD
MCDANIEL ST
PRYOR RD
SYLVAN RD
STEWART AVE
HARLAN DR
CLEVELAND AVE
JONESBORO RD
MACON DR
CONSTITUTION RD
SOUTH RIVER
CITY LIMITS
CENTRAL OF GA RY
I-20
I-75
I-85
I-285
● 50 ATLANTA AIRPORT (ESSA)
● 51 STEWART - LAKEWOOD
● 52 ATLANTA STADIUM
(O-FIREHOUSE NO.5)
● 53 GEORGIA NATIONAL GUARD
● 54 JOHNSON MOTOR LINES
● 55 GREEN BROTHERS
● ● ● ● SOUTH RIVER BASIN BOUNDARY
(00.00) TOTAL RECORDED RAINFALL, 1969
*Data Augmented From Station 51
UPPER SOUTH RIVER
DRAINAGE BASIN
RECORDING
RAINGAGE NETWORK

Figure 9



A. APPROXIMATE LOCATION OF 2-WEEK STORM
B. APPROXIMATE LOCATION OF 1-MONTH STORM
C. APPROXIMATE LOCATION OF 6-MONTH STORM

NOTE:

1. NEGLECTS VARIATIONS IN STORM DURATION.
2. BASED UPON 118 STORMS IN 1969.
3. BASED UPON DATA FROM ESSA STATION, ATLANTA AIRPORT.

DISTRIBUTION OF TOTAL RAINFALL WITHIN EACH SEPARATE OVERFLOW EVENT

Figure 10

Observed Runoff in Study Area

Measured peak flow rates resulting from three storms occurring during the period of study are shown in Table 2. Recurrence intervals were determined from rainfall records at station 5, Casplan Street, and are therefore approximate for stations 2 and 3, Boulevard and McDaniel Streets, due to nonuniformity of rainfall distribution.

TABLE 2

Storm	Recurrence Interval	Peak Flow (cfs) at Monitoring Station		
		Boulevard	McDaniel	Casplan
May 8, 1969	5 yrs	985	934	626
Nov 12, 1969	2 mos	520	333	270
June 15, 1969	2 wks	320	187	244

Flows at stations 1, 4, and 6 were recorded with insufficient frequency during storm events to justify conclusions concerning peak flow rates occurring during runoff events at these stations.

Peak runoff at Casplan Street for the five–year storm was compared with the value obtained from the Burkli–Ziegler formula, which is presently used by the City of Atlanta for storm drainage design for areas over fifty acres. The calculated runoff was 610 cfs, comparing well with the measured value of 626 cfs. However, conclusions concerning the validity of present Atlanta design criteria incorporated in this formula should be based upon more data than were obtained during this study.

Rainfall–Runoff Relationships

Determination of the relationship between rainfall and runoff for each of the subbasins was obtained by unit hydrograph analysis. The unit hydrograph represents the "system" response to a sudden pulse of one inch of rainfall over the entire basin. Three major assumptions of unit hydrograph theory are as follows:

1. Identical storms with identical antecedent conditions produce identical hydrographs.
2. Time bases of all floods caused by rainfall of equal duration are the same.

3. Assuming areal and time distribution is similar for several storms, the ordinates of each hydrograph are proportional to the volume of runoff.

These assumptions define a linear system, for which a unique unit hydrograph may be obtained for each basin. However urban areas tend to be nonlinear in their response to rainfall. Runoff flows in channels and sewers at rates which are exponential functions of the hydraulic radius and slope, as defined for instance by the Manning formula. Variations in rainfall distribution during different storms will cause changes in basin runoff response. However, response characteristics may be assumed to be linear over a limited range of inputs without causing unreasonable errors. It is therefore desirable to find a linear system that will best fit a nonlinear system over a given range.

A computer program was written to calculate the unit hydrographs, using the procedure outlined by Eagleson, et al., (1966). Least–squares analysis of the convolution equations obtained in the unit hydrograph derivation yields those equations giving the best fitting unit hydrograph. Such an analysis leads to the discrete form of the Weiner–Hopf equation:

$$\phi_{fg}(j-1) = \sum_{k=1}^{\infty} h_{opt}(k)\ \phi_{ff}(j-k) \qquad j \geqslant 1$$

Where ϕ_{fg} = cross–correlation of f with g, lag (j – 1)

ϕ_{ff} = auto–correlation, lag (j – k)

f = rainfall increments (1, 2,. . . n)

g = runoff increments (1,2,. . . m)

h = system response, or unit hydrograph increments (1,2,. . . 1)

Unit hydrograph coordinates resulting from solution of this formulation include unrealistic negative numbers. At the expense of a slight increase in unit hydrograph error, these negative coordinates may be eliminated by using linear programming techniques. The error is distributed over the resulting unit hydrograph yielding a longer time base and occasional zero values for the hydrograph ordinates. Addition of slack variables to the discrete form of the Weiner–Hopf equation yields the following linear formulation:

$$\text{Minimize } Z = \sum_{k=(m-n+1)}^{2m-n+1} h(k)$$

$$\text{Subject to} \quad \phi_{fg}(j-1) = \sum_{k=1}^{\infty} h_{opt}(k)\,\phi_{ff}(j-k) + h(m-n+1+j) \qquad j = 1,2,\ldots m$$

$$h(k) \geqslant 0$$

This method guarantees a feasible solution but it will yield slightly more error than if hydrograph coordinates were allowed to be negative. At present, this is the best way to apply a linear formulation to nonlinear data. A listing of the computer program written in Fortran IV is included in Appendix B.

Preliminary calculations with this program indicated that one-hour rainfall and runoff increments are inadequate for satisfactory hydrograph derivations. Eagleson and Shack (1966) have shown that for urban areas with runoff concentration times of approximately 20 minutes, one-minute rainfall increments are required for accurate analysis of high frequency components of the basin response. However, such accuracy was unnecessary for this study. Fifteen-minute readings proved to be of sufficient frequency to define the hydrographs for three subbasins. Rainfall contributing to flow at station 2, Boulevard, was monitored at station 52. Rainfall contributing to flow at station 5, Casplan Street, was monitored at station 51. Rainfall contributing to flow at station 3, McDaniel Street, was calculated by a weighted average of rainfall at stations 51 and 52, determined by the Thiessen method. Therefore, this study focuses upon the three subbasins for which accurate rainfall-runoff relationships may be defined. Two of these three areas contain combined sewers.

Storms monitored during the period of study were screened to ensure approximately uniform areal distribution, synchronization of rainfall and runoff records, single peaks, and recurrence intervals similar to those considered to be most significant in combined sewer pollution problems. Generally these were short, relatively intense storms yielding more than 0.5 inch of rain. Unit hydrographs were derived for each selected storm and an average unit hydrograph for each subbasin was then obtained by determining the average peak and the average time to the peak and fitting the hydrograph to this point. The alternative, more complicated methods of nonlinear analysis are considered to be unwarranted for this study.

Resultant hydrographs for area 2, contributing to overflow station 2, are shown in Figure 11, along with the average unit hydrograph. Figures 12 and 13 show the average unit hydrographs for areas 3 and 5. Each figure includes the value of the peak hydrograph ordinate, and the time to peak.

Simulation Procedure

The conventional approach to assessment of pollution from combined sewer overflows entails collection of samples for water quality analysis at equal time intervalo throughout a number of overflows. Results indicate order-of-magnitude concentrations of various parameters that are present in the overflows. Simultaneous monitoring of flow rate permits calculation of total mass discharges of these parameters to the receiving stream.

Although widely used, this approach has a number of disadvantages. Adequate personnel are required to enable sampling of water quality throughout an overflow of unpredictable commencement and duration. Automatic sampling at preset time intervals throughout the

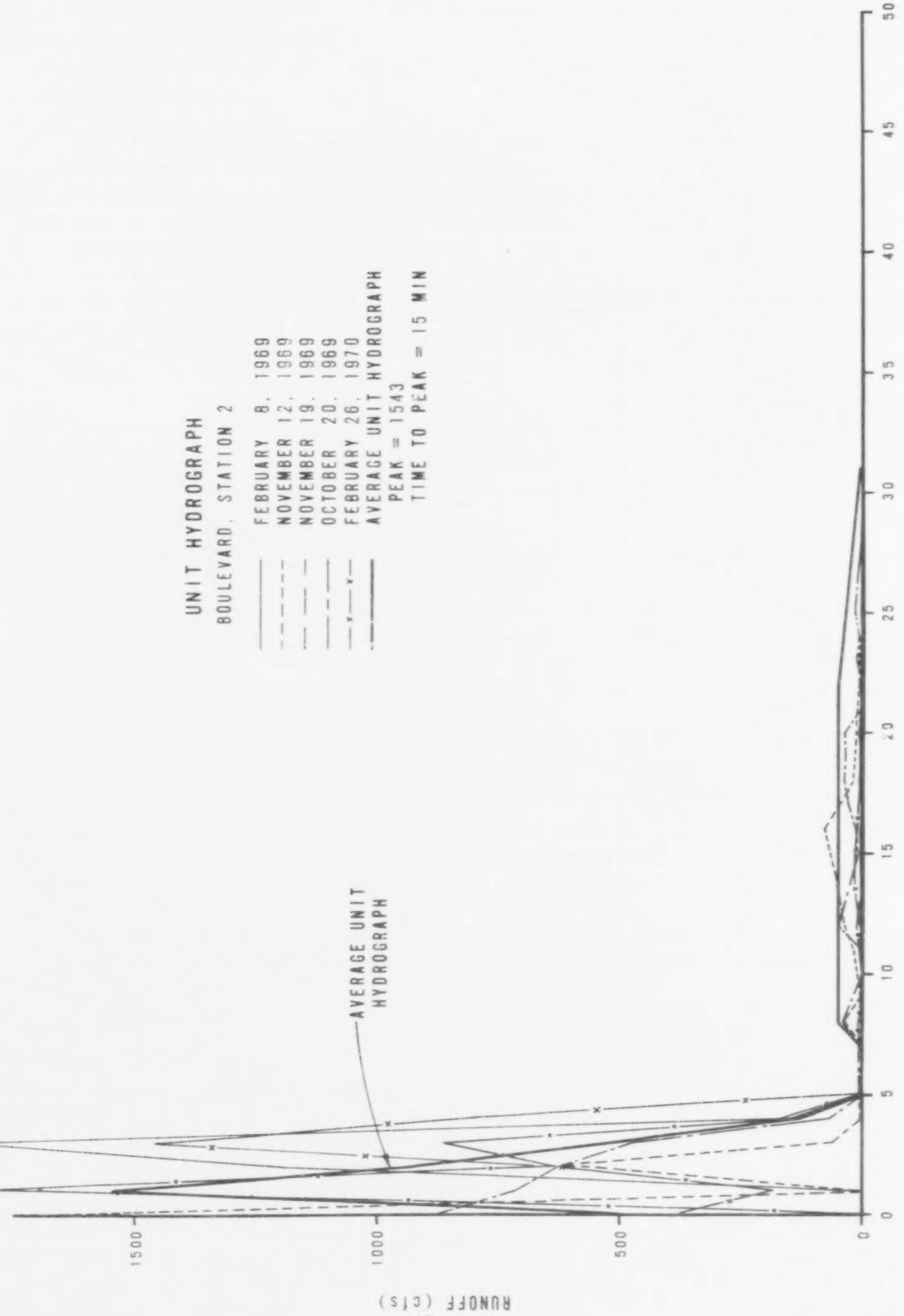

UNIT HYDROGRAPH
BOULEVARD, STATION 2
FEBRUARY 8, 1969
NOVEMBER 12, 1969
NOVEMBER 19, 1969
OCTOBER 20, 1969
FEBRUARY 26, 1970
AVERAGE UNIT HYDROGRAPH
PEAK = 1543
TIME TO PEAK = 15 MIN
AVERAGE UNIT HYDROGRAPH
RUNOFF (cfs)
0
500
1000
1500
0
5
10
15
20
25
30
35
40
45
50
1/4 - HOUR TIME INCREMENTS

Figure 11

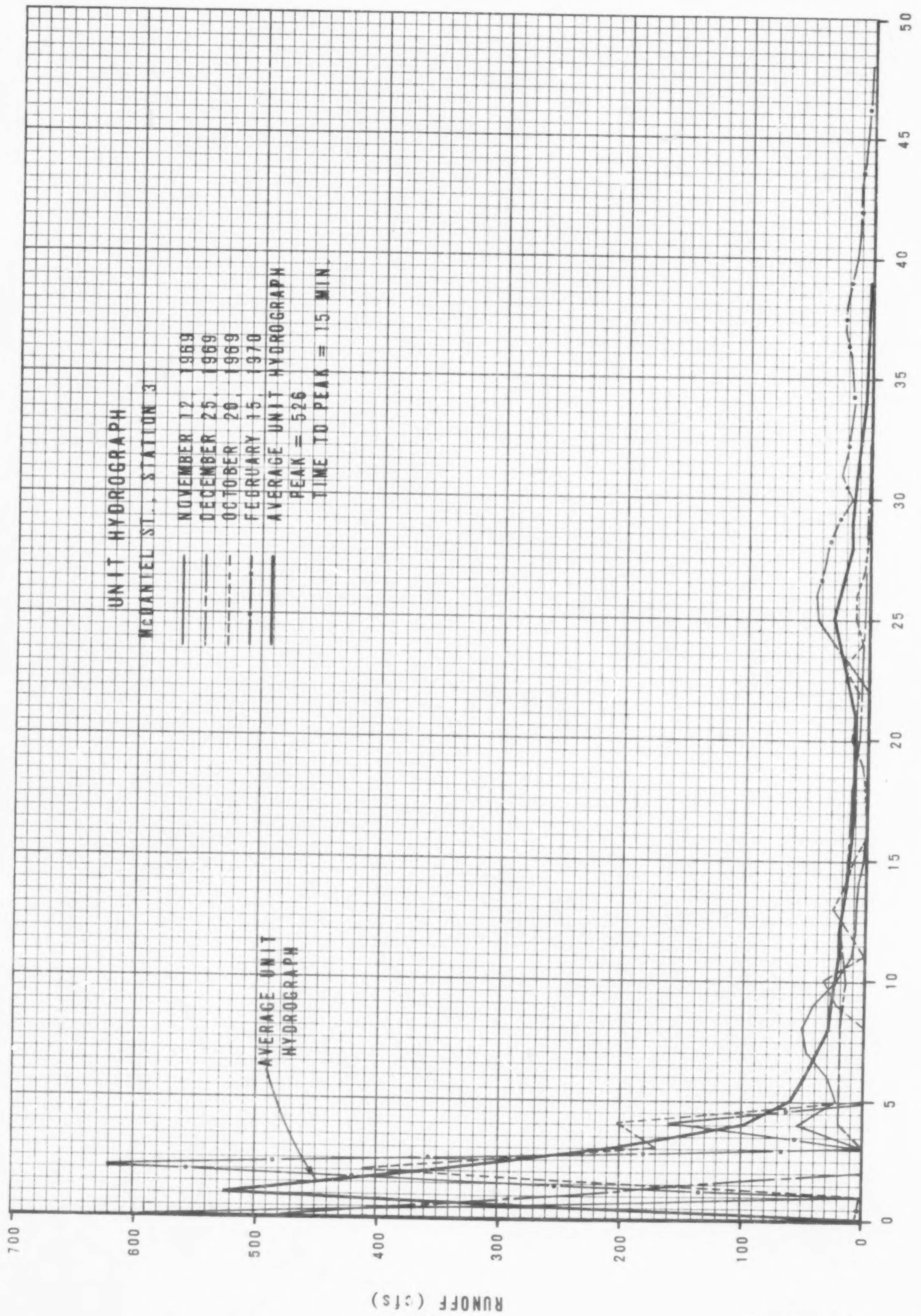

UNIT HYDROGRAPH
McDANIEL ST., STATION 3
NOVEMBER 12, 1969
DECEMBER 25, 1969
OCTOBER 20, 1969
FEBRUARY 15, 1970
AVERAGE UNIT HYDROGRAPH
PEAK = 526
TIME TO PEAK = 15 MIN.
AVERAGE UNIT HYDROGRAPH
RUNOFF (cfs)
0
100
200
300
400
500
600
700
1/4 - HOUR TIME INCREMENT
0
5
10
15
20
25
30
35
40
45
50

Figure 12

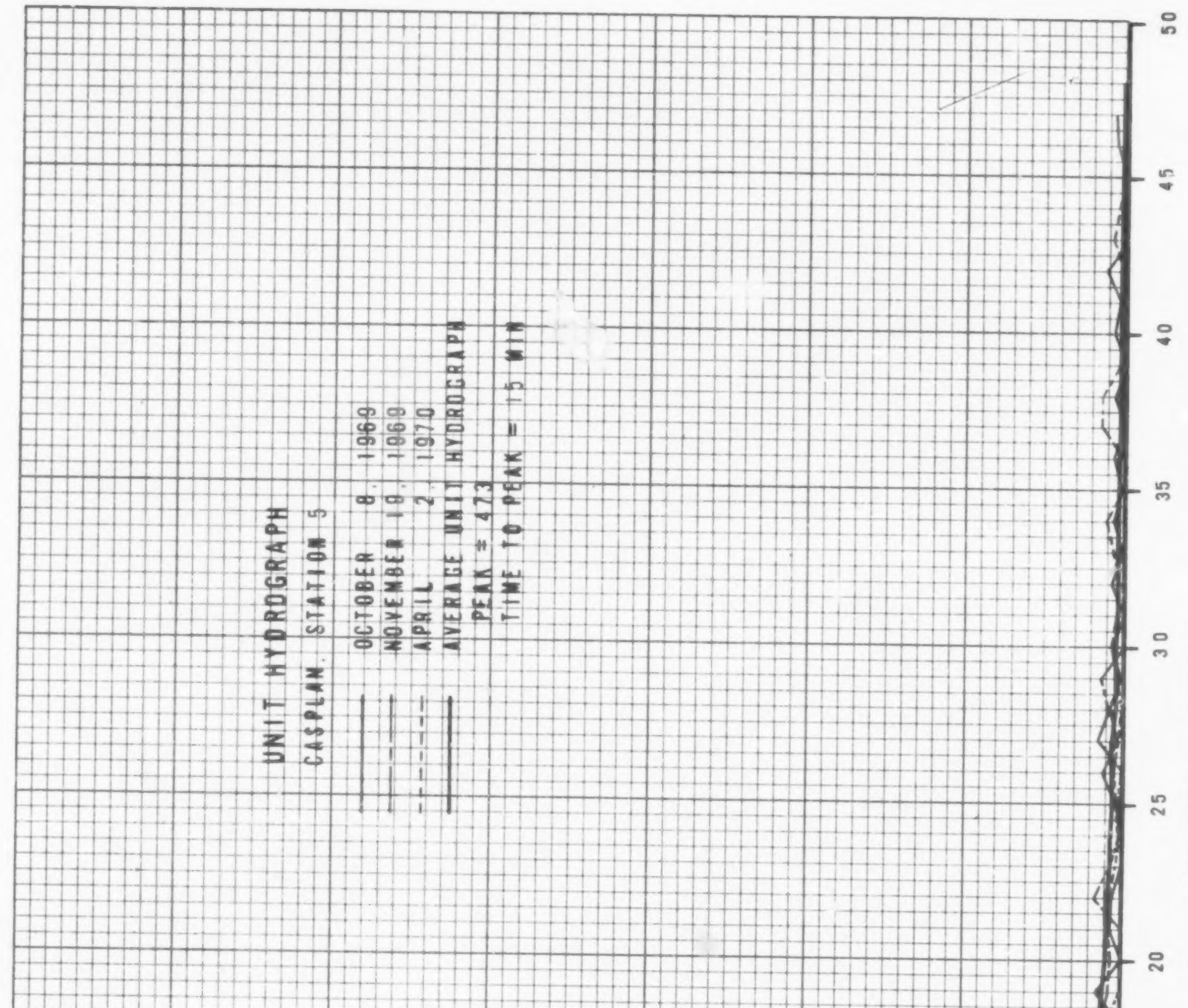

UNIT HYDROGRAPH
CASPLAN STATION 5
OCTOBER 8, 1969
NOVEMBER 19, 1969
APRIL 2, 1970
AVERAGE UNIT HYDROGRAPH
PEAK = 473
TIME TO PEAK = 15 MIN
AVERAGE UNIT HYDROGRAPH
RUNOFF (cfs)
0
100
200
300
400
500
600
1/4 - HOUR TIME INCREMENT
0
5
10
15
20
25
30
35
40
45
50

Figure 13

entire period is difficult due to the limited number of samples that may be taken. Attempts to utilize the automatic sampler for this purpose invariably yield incomplete coverage of runoff from long duration storms, or insufficient coverage from short duration storms. In addition, this approach does not permit assessment of combined sewer pollution from a wide range of storm frequencies. Conclusions are generally based upon storms occurring during a short period of study, thereby limiting observation of any significant number of low frequency storms.

Preliminary analysis of water quality data indicated the existence of a relationship between pollutant concentrations, flow rate and time for each of the three areas focused upon in this study. This observation suggested the possibility of a simulation scheme to utilize the water quality data most efficiently. A brief summary of this new approach follows.

As described previously in this section, rainfall intensity–duration–frequency curves were developed for the study area for relatively high frequency storms. The curves were utilized to synthesize a series of rainstorms. For each storm the peak hourly intensity was 2.5 times the average hourly intensity for the whole storm. In one series the peak occurred at one–quarter of the duration. In another series the peak occurred at three–quarters of the duration. For each series the following storms were synthesized:

Storm Frequency	DURATION				
(weeks)	(hours)				
2	1	2	5	–	–
4	1	2	5	10	–
26	–	2	5	10	–
52	–	2	5	10	20

The simulated rainstorms were convolved with unit hydrographs for the three areas to yield the time variation of runoff from each of the storms. Equations relating pollutant concentrations to flow rate and time were calculated from the data and were then utilized to determine concentrations and total pollutant discharges from each of the simulated storms. Resulting values are accurate within the bounds of confidence implicit in the calculation of the average unit hydrographs and the pollutant concentration equations. In general, results are considered to be reasonable average values, about which farily wide distributions occur.

Advantages of this new approach are significant. It permits utilization of automatic sampling devices without the requirement for complete coverage of a storm overflow. This decreases the number of personnel needed. The data collection program is much more simple and flexible, since discrete data points relating concentration to flow rate are more readily obtained than complete sequences of points covering an entire period of overflow. In addition to these advantages, the simulated storms may be varied in order to determine the pollution effect of both high and low frequency storms. The alternative procedure of waiting for the actual storms and monitoring the overflows is far more time–consuming. The new approach utilized in this study therefore yields considerably more information from a given data collection program than the conventional approach. The improved efficiency requires only that hourly rainfall records be available near the study area, in order to derive the intensity–duration–frequency curves.

The rainfall records were examined to determine storms of approximately uniform areal distribution. Runoff coefficients during these storms were calculated for each area and averaged. Results are shown in Table 3.

TABLE 3

Sub–basin	Type of Sewer	No. of Storms	Runoff Coefficient	Standard Deviation
1	Combined	5	0.31	0.06
2	Combined	12	0.42	0.06
3	Combined	12	0.42	0.09
4	Separate	15	0.33	0.12
5	Separate	19	0.56	0.21
6	Separate	19	0.31	0.19
		Average	0.39	

Some of the storm runoff in areas served by combined sewers is intercepted by the conduits that normally carry dry weather sewage flow to the wastewater treatment plants. For large storms this fraction is very small; however for small storms this can amount to a significant part of the total runoff. Since intercepted storm flows are generally bypassed to the receiving stream at the treatment plants, their total volumes were added to overflow volumes prior to calculation of runoff coefficients.

SECTION V

COMBINED SEWER OVERFLOWS

Major Overflows in Study Area

Preliminary work on the project established the existence of three major overflow points within the study area. All three occur at the ends of combined sewers draining highly developed and densely populated areas. Their location is shown in Figure 14.

Largest of all is the overflow at Boulevard, based on area and population served, as well as on volumes discharged. Overflows from the Boulevard and Confederate Avenue combined sewers are discharged into Intrenchment Creek. The third major overflow, at McDaniel Street, contributes wet weather flows to the north branch of South River. Table 4 summarizes areas and populations served by these major combined sewer systems down to the points of overflow.

TABLE 4

Station Number	Station Designation	Acres Drained	Population[1] in Area	Receiving Stream
1	Confederate Ave	1129.02	12,250	Intrenchment Creek
2	Boulevard	2421.45	40,250	Intrenchment Creek
3	McDaniel Street	967.62	12,750	North Branch of South River
		4518.09	**65,250**	

[1] Based on population estimates for census tracts and tract zones, and population distribution studies by ARMPC (Atlanta Region Metropolitan Planning Commission).

Approximately 2,400 feet downstream of the McDaniel Street overflow, at the Joyland Park area, a smaller (66–inch diameter) combined sewer also discharges in to the north branch. It serves a total drainage area of approximately 194 acres. This is not considered a major overflow, however, since a separate system serves the largest fraction of the drainage area and sanitary sewage from only a small part of it enters the combined sewer trunk.

Confederate Avenue Overflow

The major combined sewer discharging south of Confederate Avenue is also known as Stockade Trunk and Lester Street sewer. It consists of a 10' x 10' concrete arch cross

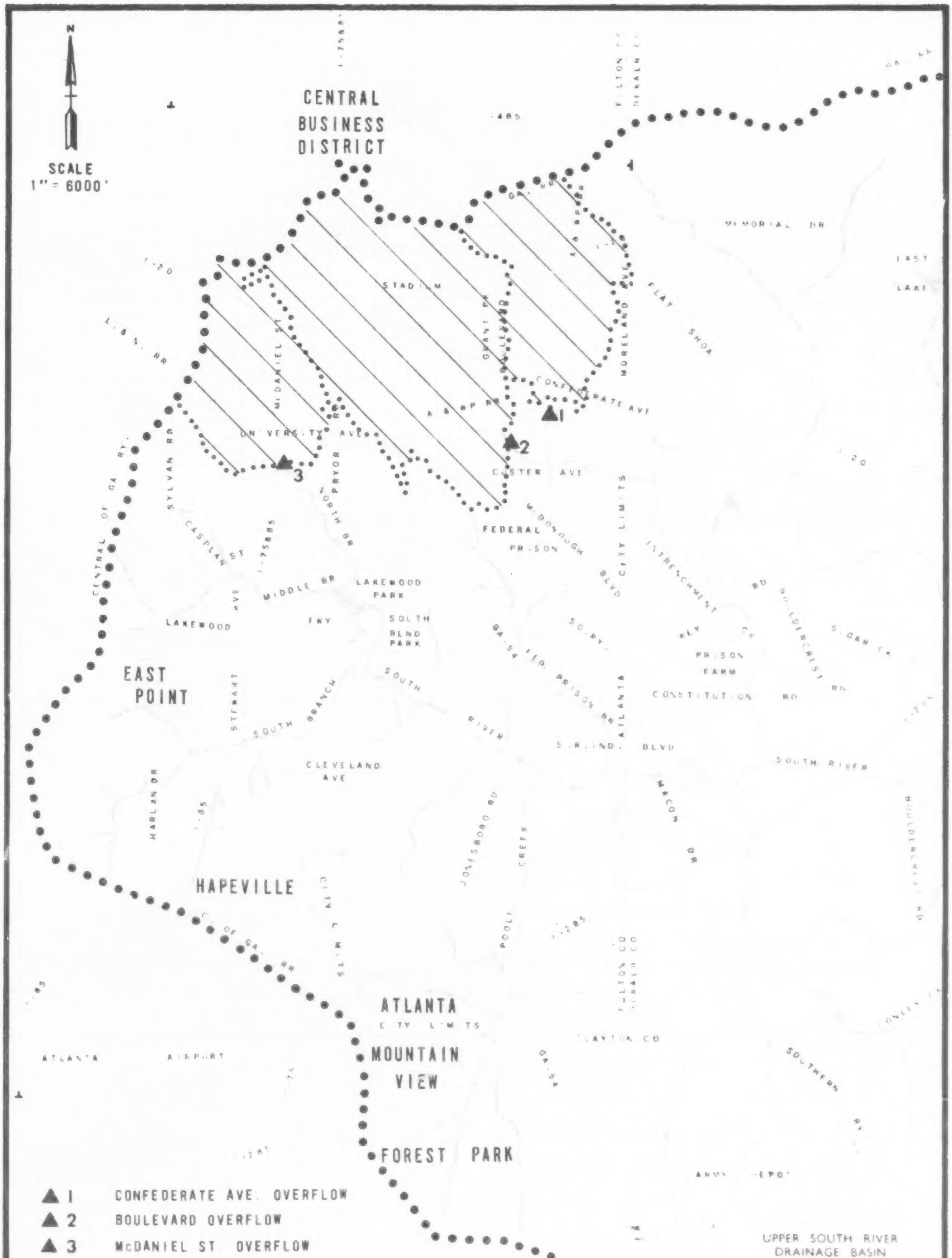

N
SCALE
1" = 6000'
CENTRAL
BUSINESS
DISTRICT
STADIUM
McDANIEL ST
GRANT PK
BOULEVARD
MORELAND AVE
CONFEDERATE AVE
UNIVERSITY AVE
CUSTER AVE
FEDERAL PRISON
McDONOUGH BLVD
CITY LIMITS
MEMORIAL DR
EAST LAKE
FLAT SHOALS
I-20
I-75&85
SYLVAN RD
NORTH BR
LAKEWOOD PARK
SOUTH BEND PARK
LAKEWOOD FWY
EAST POINT
HAPEVILLE
ATLANTA
CITY LIMITS
MOUNTAIN VIEW
FOREST PARK
ATLANTA AIRPORT
SOUTH RIVER
CLEVELAND AVE
JONESBORO RD
POOLE CREEK
MACON DR
CONSTITUTION RD
PRISON FARM
INTRENCHMENT CK
SUGAR CK
BOULDERCREST RD
CLAYTON CO
FULTON CO
DEKALB CO
ARMY DEPOT
SOUTHERN RY
CENTRAL OF GA RY
▲1 CONFEDERATE AVE. OVERFLOW
▲2 BOULEVARD OVERFLOW
▲3 McDANIEL ST. OVERFLOW
······· CONTRIBUTORY AREA BOUNDARY
● ● ● ● SOUTH RIVER BASIN BOUNDARY
UPPER SOUTH RIVER
DRAINAGE BASIN
LOCATION OF MAJOR
COMBINED SEWER OVERFLOWS

Figure 14

section with an almost flat, V–shaped concrete bottom. Center line elevation of the bottom slab is approximately 6 inches lower than the sides. Steep longitudinal slopes in this sewer induce considerable velocities, especially at high flow conditions. Maximum recorded velocity at this station was 15 feet per second. Slopes vary from section to section. A total fall of 28.3 feet takes place in the last 4,130 feet of the sewer. The straight section ending at the overflow point falls 3.0 feet in a length of 630 feet, for a slope of 0.476 percent.

Details of this sewer at the regulator device can be seen in Figures 15 and 16. Separation of dry weather flows is obtained by means of an opening on the side wall of the sewer, a depressed area in the bottom in front of the opening, and a small weir or dam across the sewer just downstream of it. Entrance to the interceptor is protected by a bar screen located outside and adjacent to the sewer on the east side. The bar screen is installed within an open pit having concrete bottom and brick walls. The first section of the interceptor, only 31 feet long, is made of 20–inch diameter vitrified clay pipe. At the first manhole a change to 27–inch diameter concrete pipe takes place. At a second manhole, located only 29 feet downstream, diameter increases to 30 inches. The interceptor flows south on the east side of the stream for approximately 2,200 feet, until it meets the Intrenchment Creek interceptor.

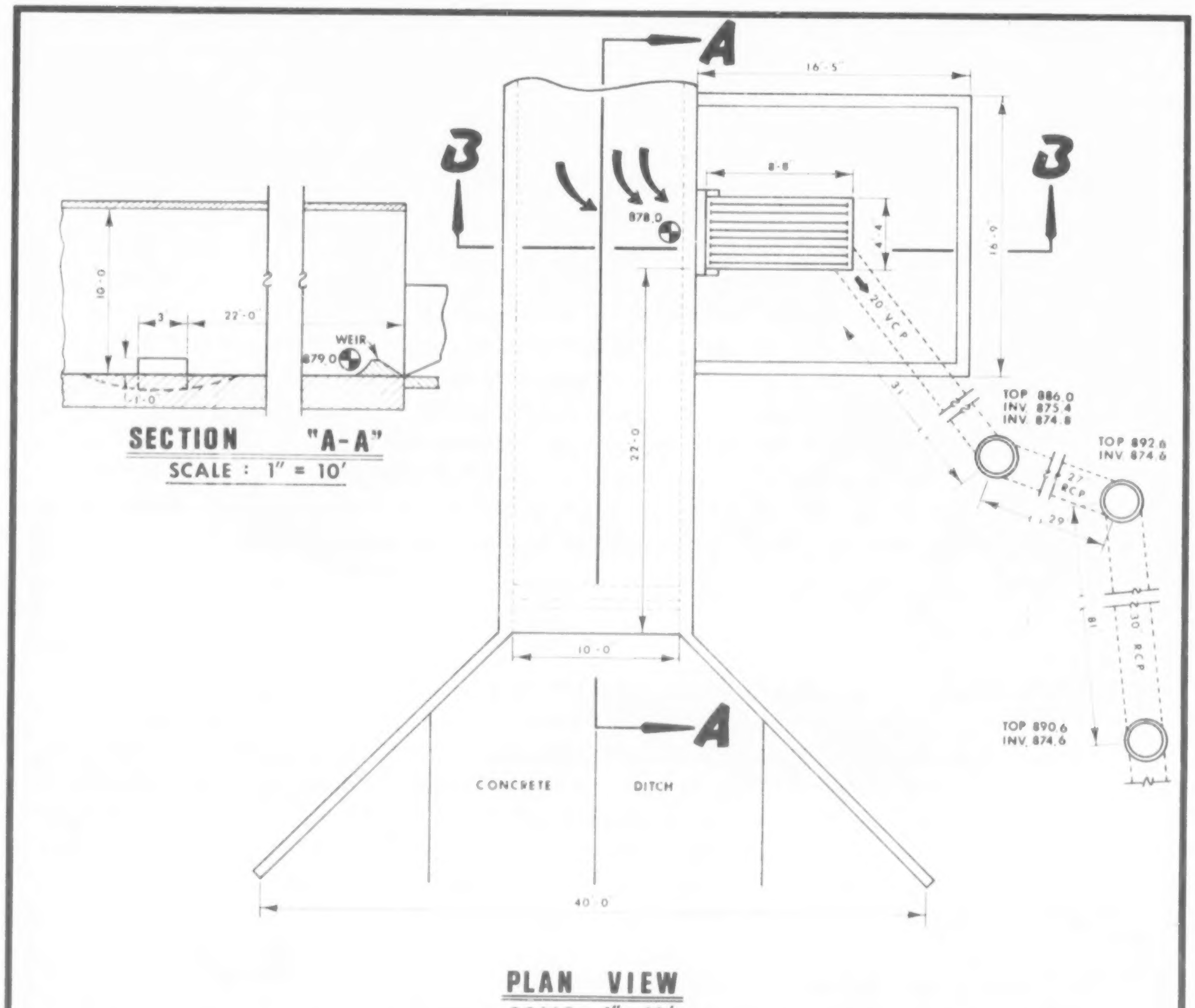
Boulevard Overflow

At the location of the regulator device, the Boulevard combined sewer consists of a double 9' x 13' rectangular sewer, built of reinforced concrete. The regulator consists of a 4–foot–wide, semicircular bottom trough extending from side to side of the combined sewer in a direction perpendicular to the flow, and protected by a full–width bar screen. A concrete dam or weir located on the downstream side of the bar screen to intercept dry weather flows, as can be seen in Figure 17. Crest of this weir is about one foot higher than the top of the bar screen. Below this point the main sewer has been extended 212 feet to the point of discharge, where Intrenchment Creek begins.

The interceptor originating at the regulator device is known as the Intrenchment Creek interceptor. Construction plans for this structure are dated 1913, and call for a 48–inch vitrified clay pipe beginning at the regulator. This pipe curves rapidly to parallel the creek on the north side. The interceptor receives large amounts of grit, requiring frequent cleaning through an adjacent brick structure constructed later. Inspection of the first three manholes downstream of this structure indicated large grit accumulation such that only three feet of interceptor depth were available for flow. A reverse bottom slope exists between the brick structure and the first three manholes. Separate sanitary sewers enter the Intrenchment Creek interceptor throughout its route. About 2,300 feet downstream of the regulator it receives the 30–inch diameter Confederate Avenue interceptor.

Photographs in Figure 18 show the present end of the sewer, where the creek originates, and a flow level recorder installed in a manhole at the interceptor.

Longitudinal slopes vary along the double 9' x 13' section of the combined sewer. A total fall of 12.5 feet takes place in a distance of 3,780 feet from the invert of the manhole

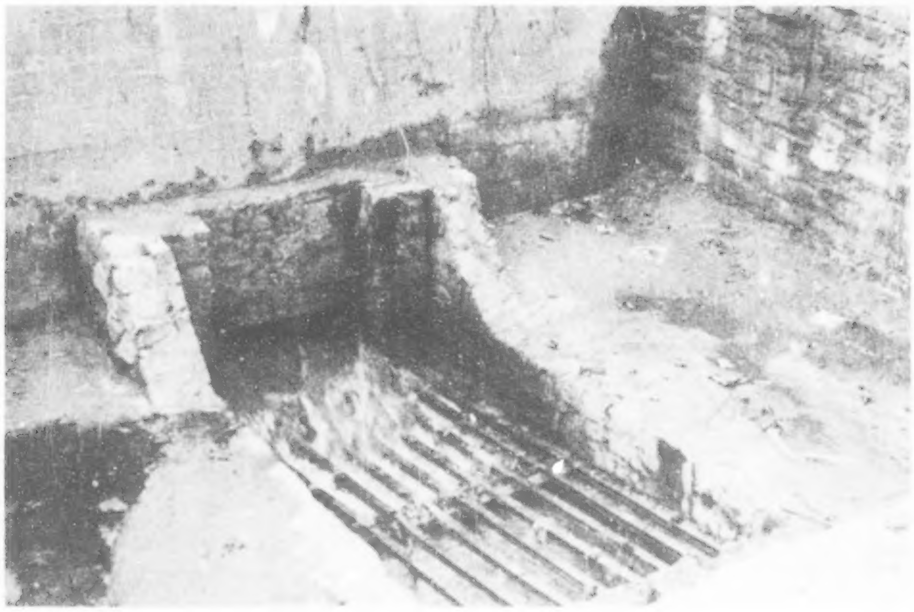

A
16'-5"
B
B
8'-8"
878.0
4'-4"
16'-9"
20" V.C.P.
31'
TOP 886.0
INV. 875.4
INV. 874.8
TOP 892.8
INV. 874.6
27" RCP
29'
81'
30" RCP
TOP 890.6
INV. 874.6
10'-0"
3'
22'-0"
WEIR
879.0
1'-1"-0"
SECTION "A-A"
SCALE : 1" = 10'
22'-0"
10'-0"
A
CONCRETE DITCH
40'-0"
PLAN VIEW
SCALE : 1" = 10'
FILL
884.6
884.2
10'-0"
884.7
884.3
5'-6"
WEIR
879.3
879.0
878.1
879.4
END VIEW
SCALE : 1" = 10'
10'-0"
BAR SCREEN
879.0
878.9
879.7
877.9
20" V.C.P. INTERCEPTOR
876.9
SECTION "B-B"
SCALE : 1" = 10'

GENERAL DIMENSIONS

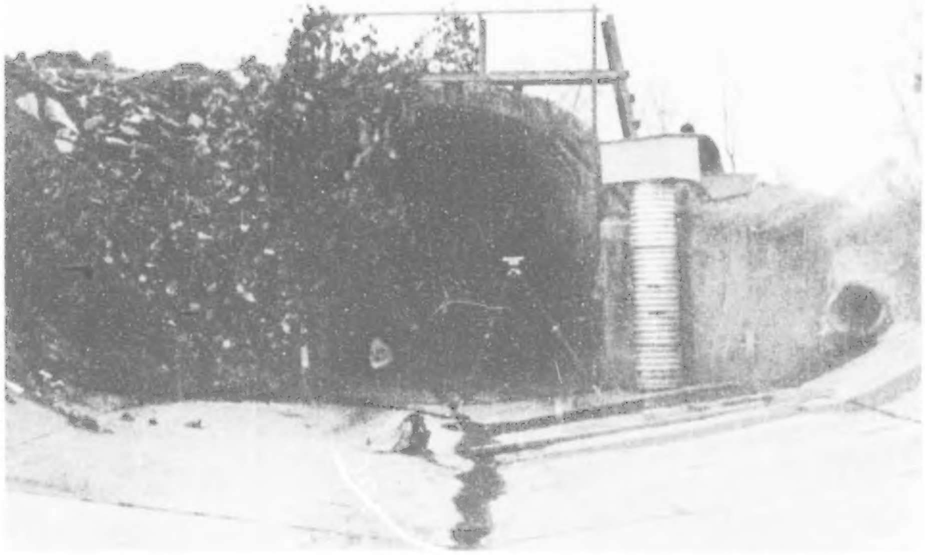
CONFEDERATE AVENUE
COMBINED SEWER
OVERFLOW

Figure 15

Overflow
Sampling



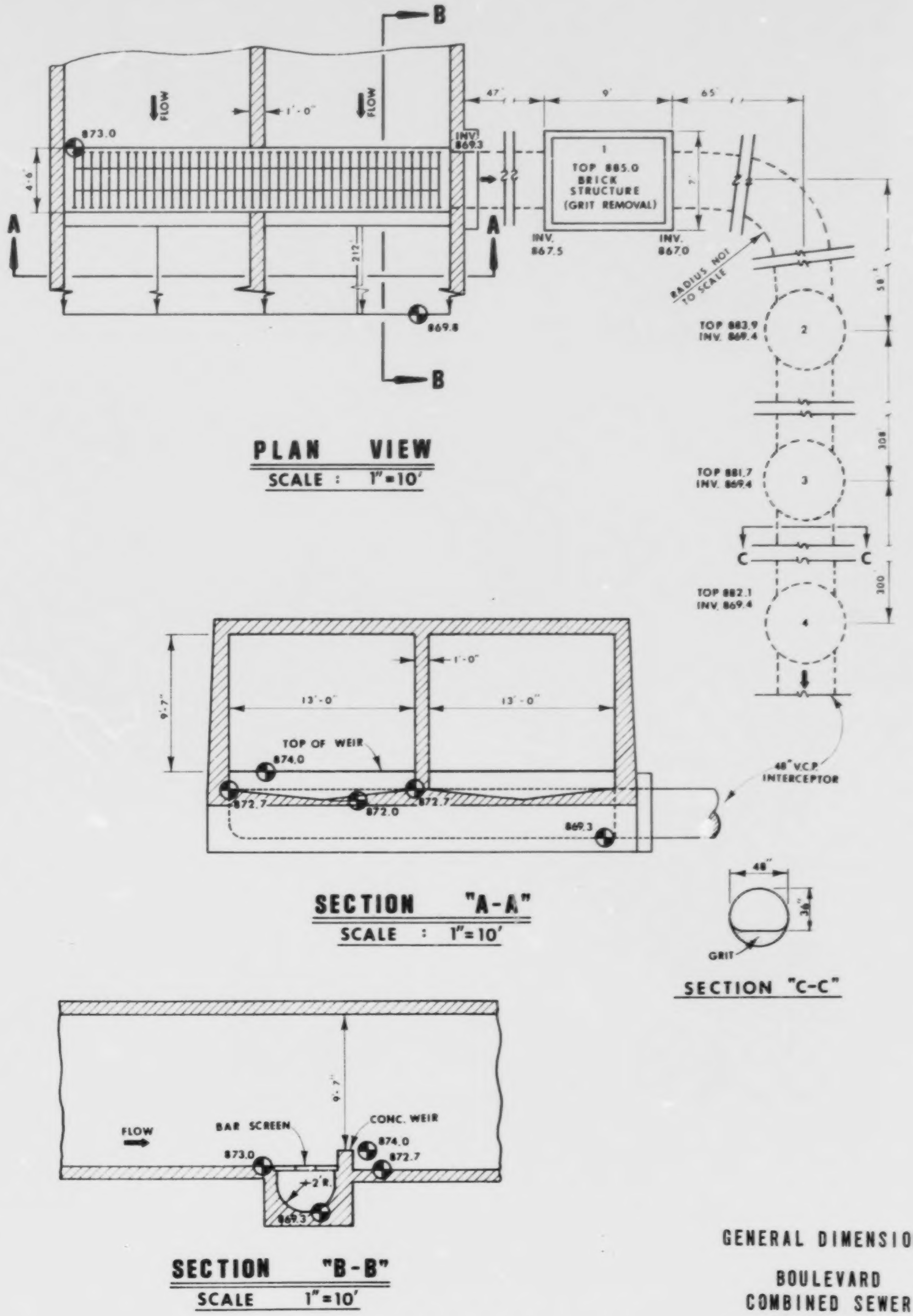
Regulator Device
and Bar Screen
at Interceptor



CONFEDERATE AVENUE
COMBINED SEWER
OVERFLOW

Overflow Point and Flow Level Recorder

Figure 16



PLAN VIEW

SCALE : 1"=10'

SECTION "A-A"

SCALE : 1"=10'

SECTION "C-C"

SECTION "B-B"

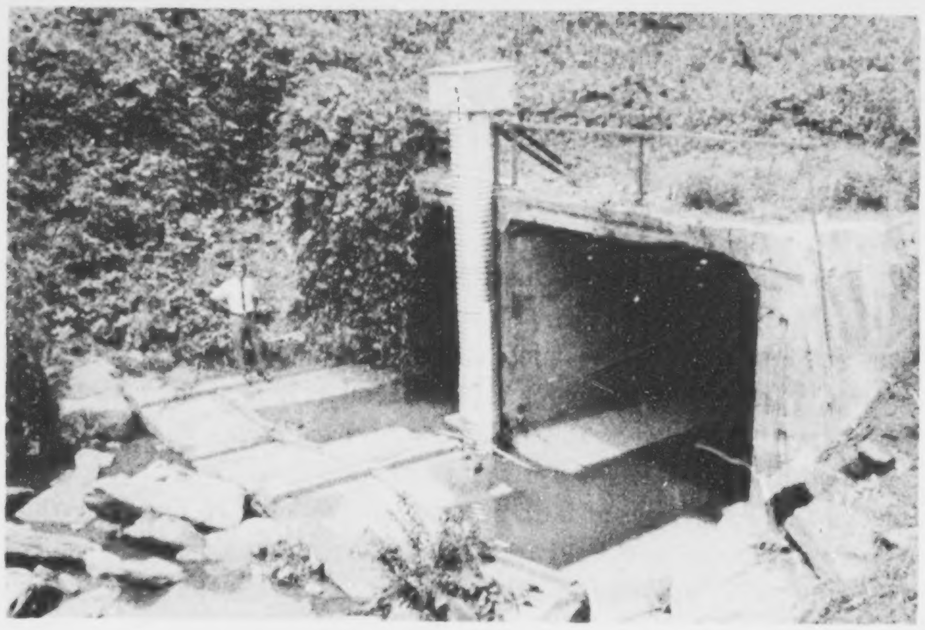
SCALE 1"=10'

B
B
A
A
FLOW
FLOW
1'-0"
873.0
4'-6"
212'
869.8
INV. 869.3
47'
9'
65'
1
TOP 885.0
BRICK STRUCTURE
(GRIT REMOVAL)
7'
INV. 867.5
INV. 867.0
RADIUS NOT TO SCALE
58'
TOP 883.9
INV. 869.4
2
308'
TOP 881.7
INV. 869.4
3
C
C
300'
TOP 882.1
INV. 869.4
4
48" V.C.P. INTERCEPTOR
9'-7"
1'-0"
13'-0"
13'-0"
TOP OF WEIR
874.0
872.7
872.0
872.7
869.3
48"
36"
GRIT
FLOW
BAR SCREEN
CONC. WEIR
9'-7"
873.0
874.0
872.7
2'R.
869.3

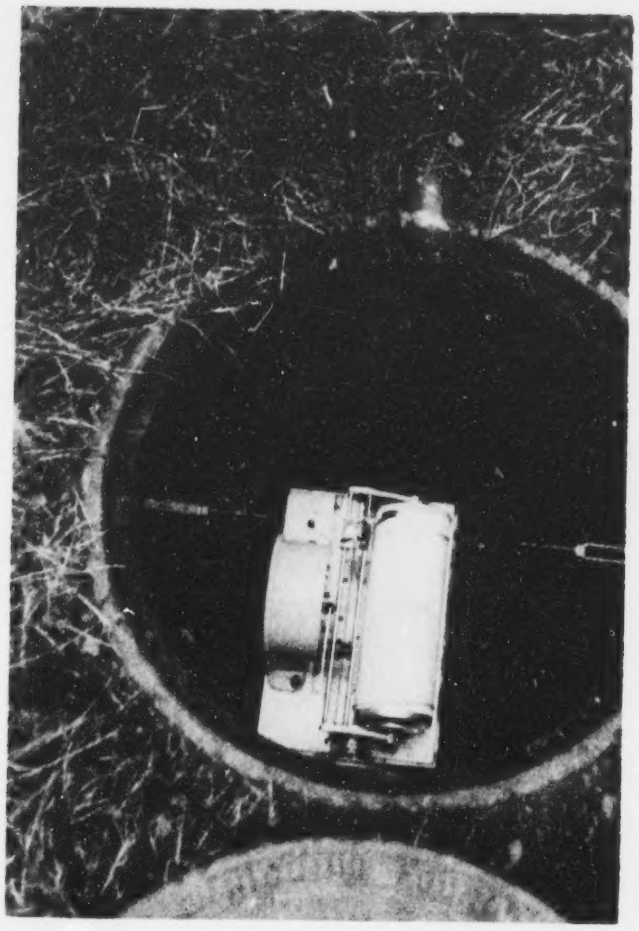
GENERAL DIMENSIONS

BOULEVARD
COMBINED SEWER
OVERFLOW

Figure 17



Overflow Point and Flow Level Recorder



Flow Level Recorder in Manhole at Interceptor

BOULEVARD
COMBINED
SEWER
OVERFLOW

Figure 18

located at Hill Street and the top of the bar screen structure at the regulator device. The straight section just upstream of the regulator slopes at approximately 0.5 percent for a length of 1,600 feet. The last section, extending 212 feet below the weir at the regulator, slopes at 1.04 percent to the present point of discharge at the beginning of Intrenchment Creek.

McDaniel Street Overflow

The McDaniel Street combined sewer has the smallest drainage area of the three major overflows studied. At its present end it consists of a 16' x 16' concrete arch culvert running for 170 feet under and across the tracks of the Atlanta and West Point Railroad. Two combined sewers join to enter this section at its upstream end. One is an 8' x 8' box culvert that comes from McDaniel Street, and slopes at 0.68 percent for the last 570 feet; the other is a 6' x 6' arch sewer entering from the west, and averaging 0.72 percent slope over the last 2,340 feet. Two smaller storm sewers serving the adjacent area also enter the trunk at this junction.

Dimensional drawings and photographs of the regulator and overflow appear in Figures 19, 20, and 21. The regulator consists of an opening broken through a concrete wall on one side, and a raised bottom at the discharge end of the culvert. A slightly depressed area in front of the wall opening facilitates entrance of dry weather flows to the interceptor.

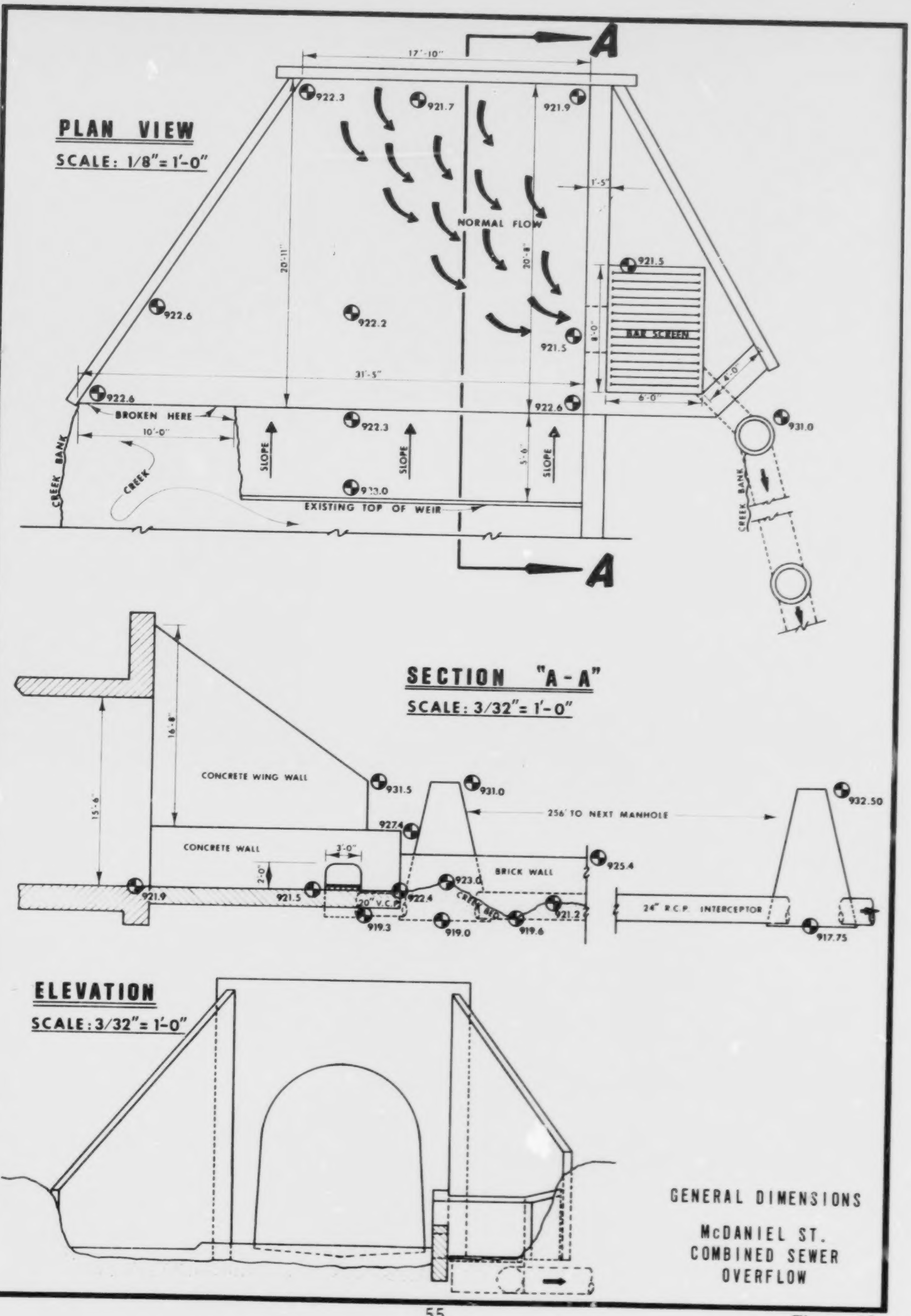
A bar screen protects the interceptor from larger debris in the combined sewage. The interceptor begins as a 20–inch pipe between the bar screen and an adjacent manhole. At this point diameter is increased to 24 inches. It continues downstream along the northeast side of the creek towards the South River treatment plant.

For convenience, overflow discharges from this sewer were measured at the 12' x 12' culvert at Manford Road, located 250 feet downstream of the overflow point. This causes a slight increase in tributary drainage area. Flows in the interceptor were also monitored at a manhole on the center line of Manford Road, next to the culvert. It was not possible to find additional manholes between this point and the south expressway, to verify invert elevations for comparative flow estimation.

At the Manford Road manhole, surcharge has been observed to occur in the interceptor at rather small wet weather flows.

Overflow Characteristics

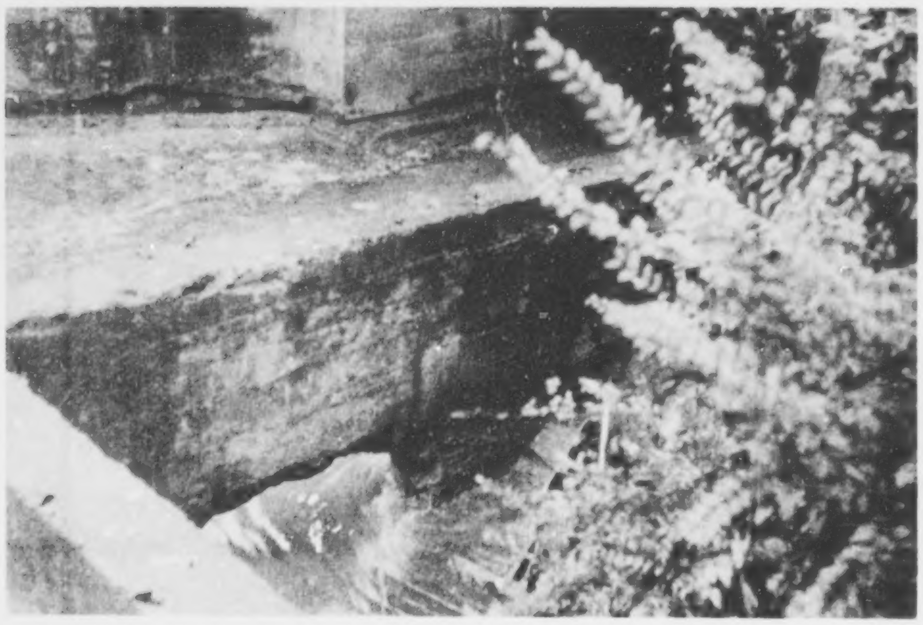
Overflows at Confederate Avenue, Boulevard and McDaniel Street were monitored for pollutant concentrations, as discussed in Section III. With a maximum of 24 bottles available from the automatic sampler, collection intervals were varied during the study period. By manually activating the samplers, short intervals of one to five minutes were


PLAN VIEW
SCALE: 1/8" = 1'-0"
A
17'-10"
922.3
921.7
921.9
1'-5"
NORMAL FLOW
20'-11"
20'-8"
921.5
922.6
922.2
8'-0"
BAR SCREEN
921.5
4'-0"
31'-5"
922.6
922.6
6'-0"
BROKEN HERE
10'-0"
922.3
931.0
SLOPE
SLOPE
5'-0"
SLOPE
CREEK BANK
CREEK
923.0
EXISTING TOP OF WEIR
CREEK BANK
A
SECTION "A-A"
SCALE: 3/32" = 1'-0"
16'-8"
15'-6"
CONCRETE WING WALL
931.5
931.0
256' TO NEXT MANHOLE
932.50
927.4
CONCRETE WALL
3'-0"
2'-0"
BRICK WALL
925.4
921.9
921.5
20" V.C.P.
922.4
923.0
CREEK BED
921.2
24" R.C.P. INTERCEPTOR
919.3
919.0
919.6
917.75
ELEVATION
SCALE: 3/32" = 1'-0"

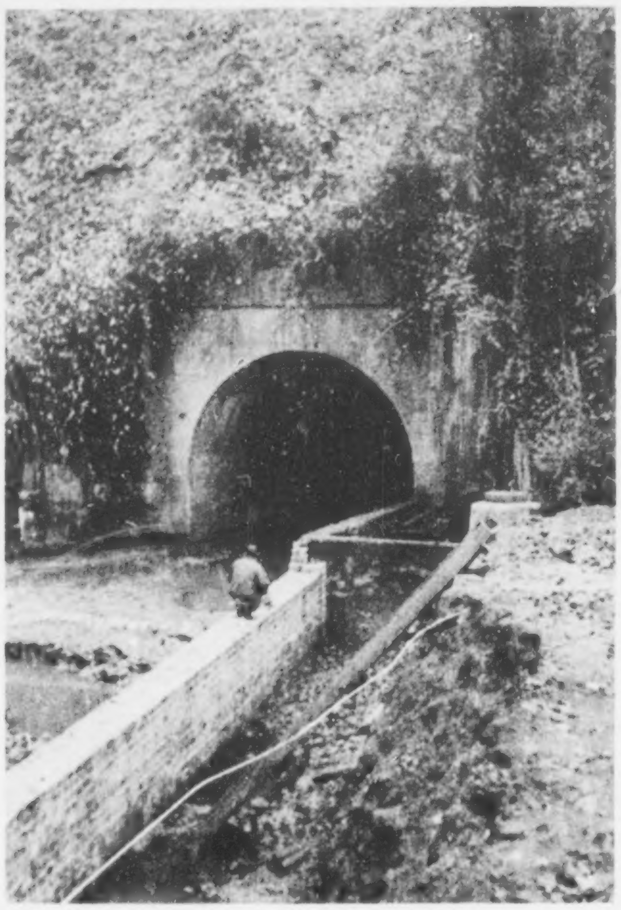
GENERAL DIMENSIONS

McDANIEL ST.
COMBINED SEWER
OVERFLOW

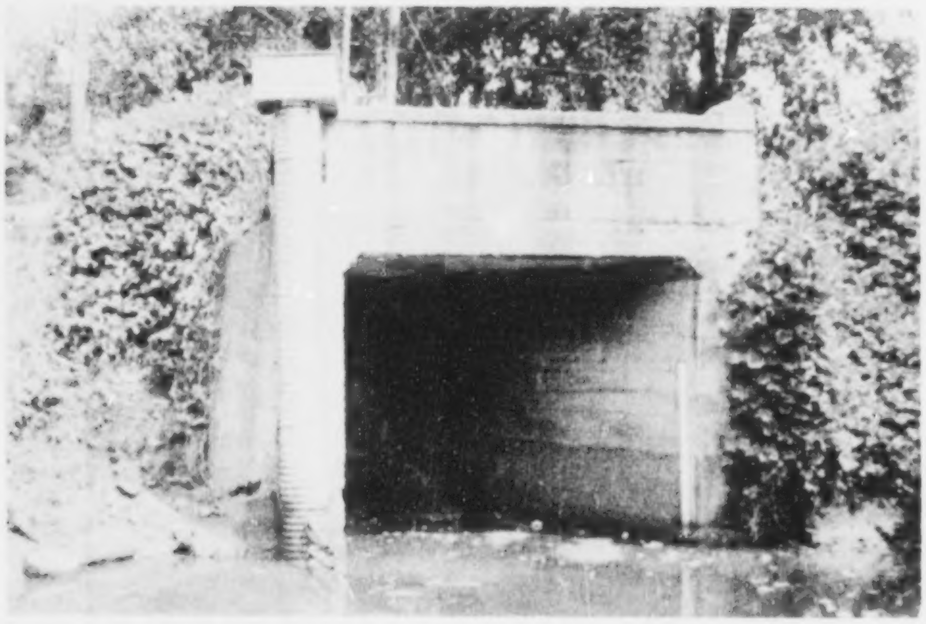
Figure 19



Regulator Device and
Bar Screen at Interceptor



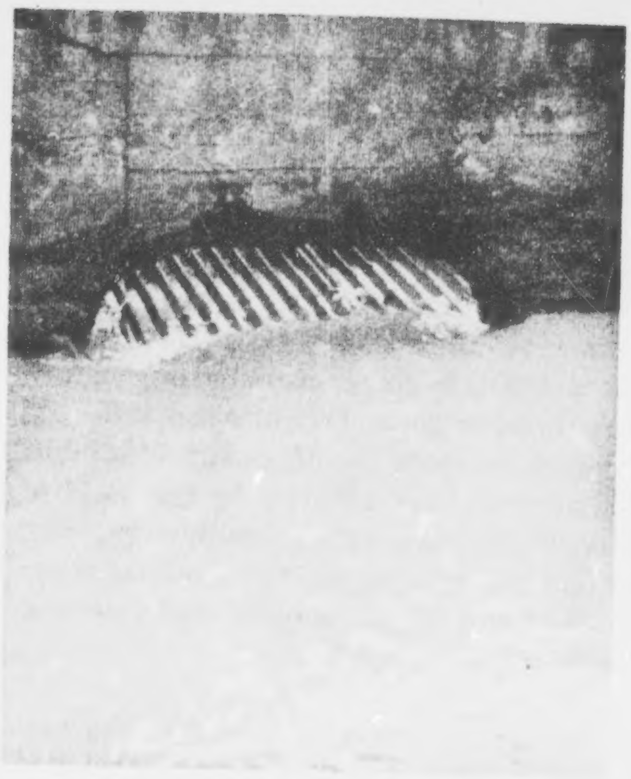
Overflow Point



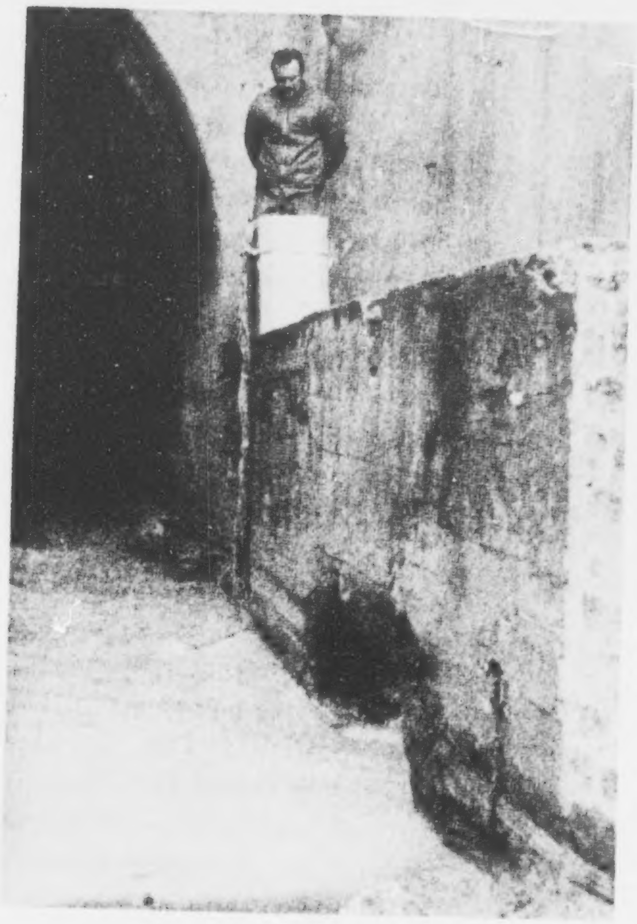
Gaging Station
Culvert Below Overflow Point

MCDANIEL STREET
COMBINED SEWER OVERFLOW

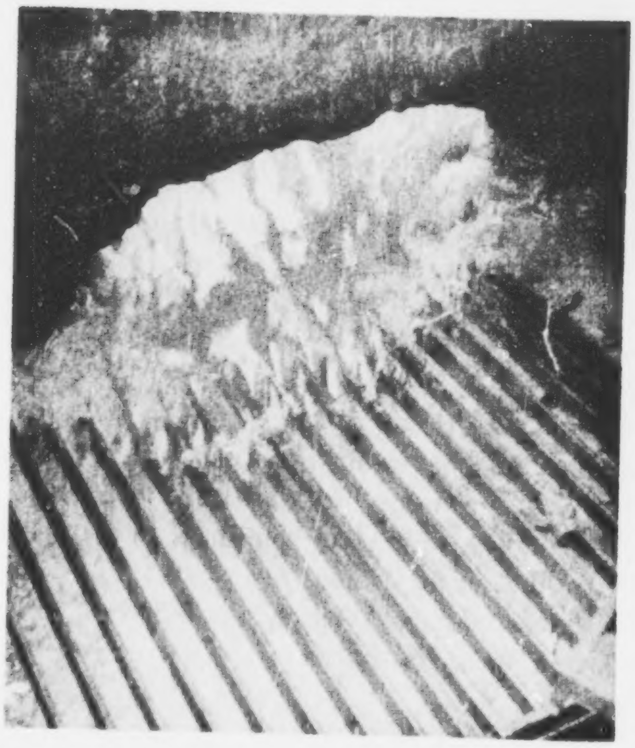
Figure 20



Opening on Side Wall
Serving as Regulator



Sampling Dry Weather Flow



Bar Screen at Interceptor

McDANIEL STREET
COMBINED SEWER OVERFLOW

Figure 21

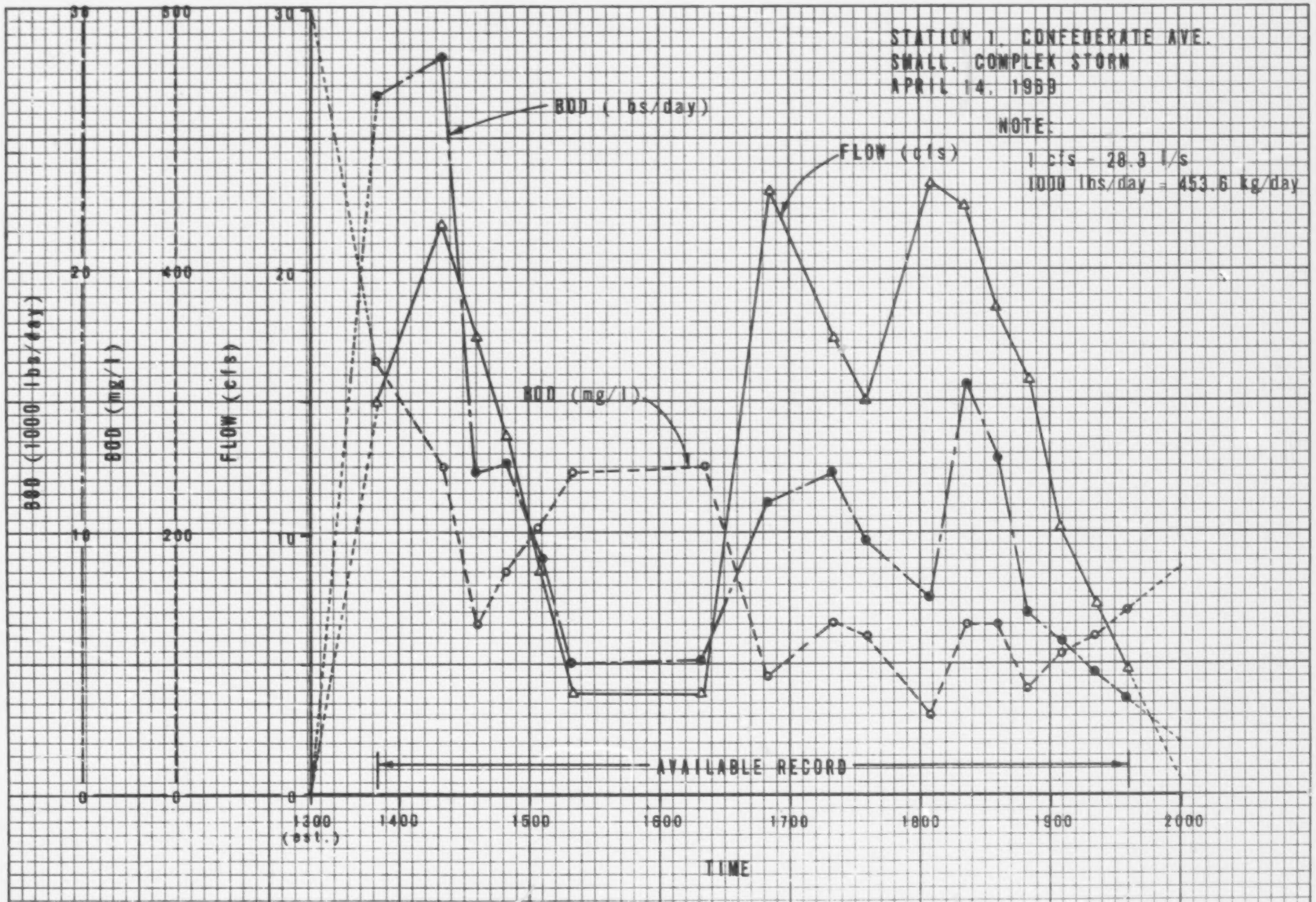
employed to define adequately concentrations near the beginning of overflows of some storms. Longer intervals of fifteen minutes or more were used to cover the entire duration of other storms.

Of the many parameters monitored, the most significant in its impact upon the environment is biochemical oxygen demand (BOD). This parameter is therefore emphasized more strongly than the others in subsequent discussions.

BOD in combined sewer overflows comes from three primary sources: normal dry weather flow during the storm, overland storm drainage, and BOD in the system at the beginning of storm runoff, including settled solids. In the South River Basin, City of Atlanta, the last component is forced out of the system in front of the storm runoff, causing high initial BOD's that decay with time as the whole system becomes tributary to the point of overflow. During the remainder of the overflow, BOD concentrations vary inversely with flow rate, indicating dilution with sewage flow entering the system. BOD from the storm drainage component and from areas served by storm sewers varies directly with flow rate. These relationships are illustrated for two storms in Figures 22 and 23.

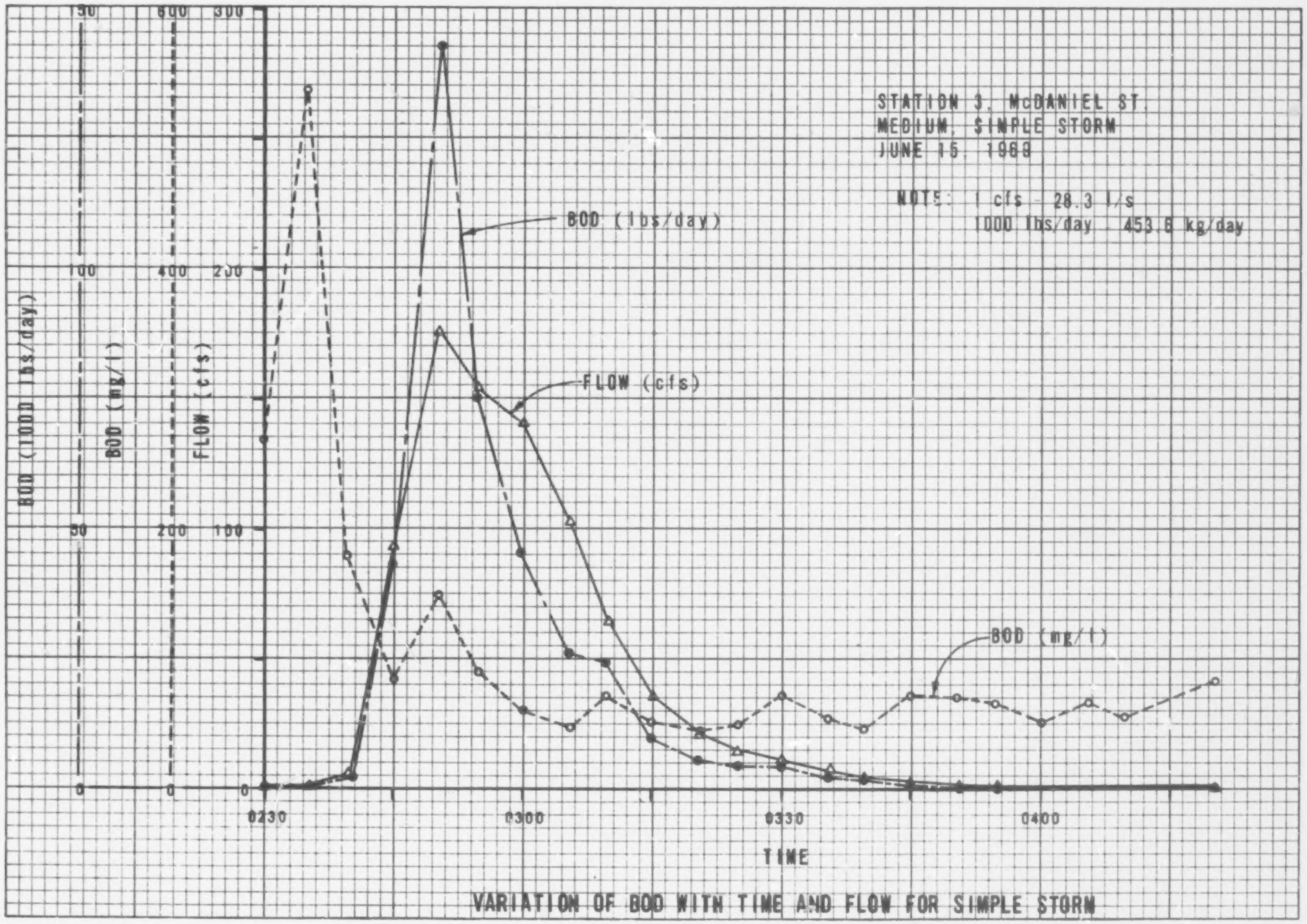
The extensive BOD data were analyzed to separate the three primary components. The wide variations from one storm to the next in recorded BOD concentrations during the dilution phase of overflows indicate the effects of many factors, including overflow rate, the duration of preceding dry weather period, the precision of laboratory analyses and sewage flow rate at the time of overflow. The last factor is very significant since the storm flows dilute dry weather sewage flows to yield the overflow BOD concentration. Hourly and daily dry weather sewage flow variations at the interceptors were monitored during the study and were utilized in conjunction with monthly sewage treatment plant records to obtain multiplication factors by means of which dry weather sewage flows at any time during the year could be compared to those occurring during an average hour of the year. With these multiplication factors, recorded BOD concentrations during overflows were adjusted to values that would have been obtained during an average hour of the year. This procedure was followed in order to improve the validity of equations relating BOD to overflow rate, without specific consideration of the time of year. The equations can be utilized to estimate BOD concentrations and pollution loads from storms occurring at particular times of the year by multiplying with the appropriate factor.

All BOD values were screened in order to separate high concentrations occurring at the beginning of overflows due to scouring of solids. Concentrations due to storm runoff alone, as determined from data at Station 5, Casplan Street, were subtracted from the data in order to leave a residual concentration attributable to dilution of influent dry weather flow. A regression analysis was then performed to determine the relationship between this concentration and flow rate. The results are shown in Table 5 as follows:



VARIATION OF BOD WITH TIME AND FLOW FOR COMPLEX STORM

Figure 22



STATION 3, McDANIEL ST.
MEDIUM, SIMPLE STORM
JUNE 15, 1969

NOTE: 1 cfs = 28.3 l/s
1000 lbs/day = 453.6 kg/day

VARIATION OF BOD WITH TIME AND FLOW FOR SIMPLE STORM

Figure 23

TABLE 5

Station Number	Station Designation	Number of Overflows	F	r^2	Statistical Significance
1	Confederate Ave	32	01.11	0.034	None(1)
2	Boulevard	97	24.55	0.205	.995+
3	McDaniel St	97	23.44	0.198	.995+

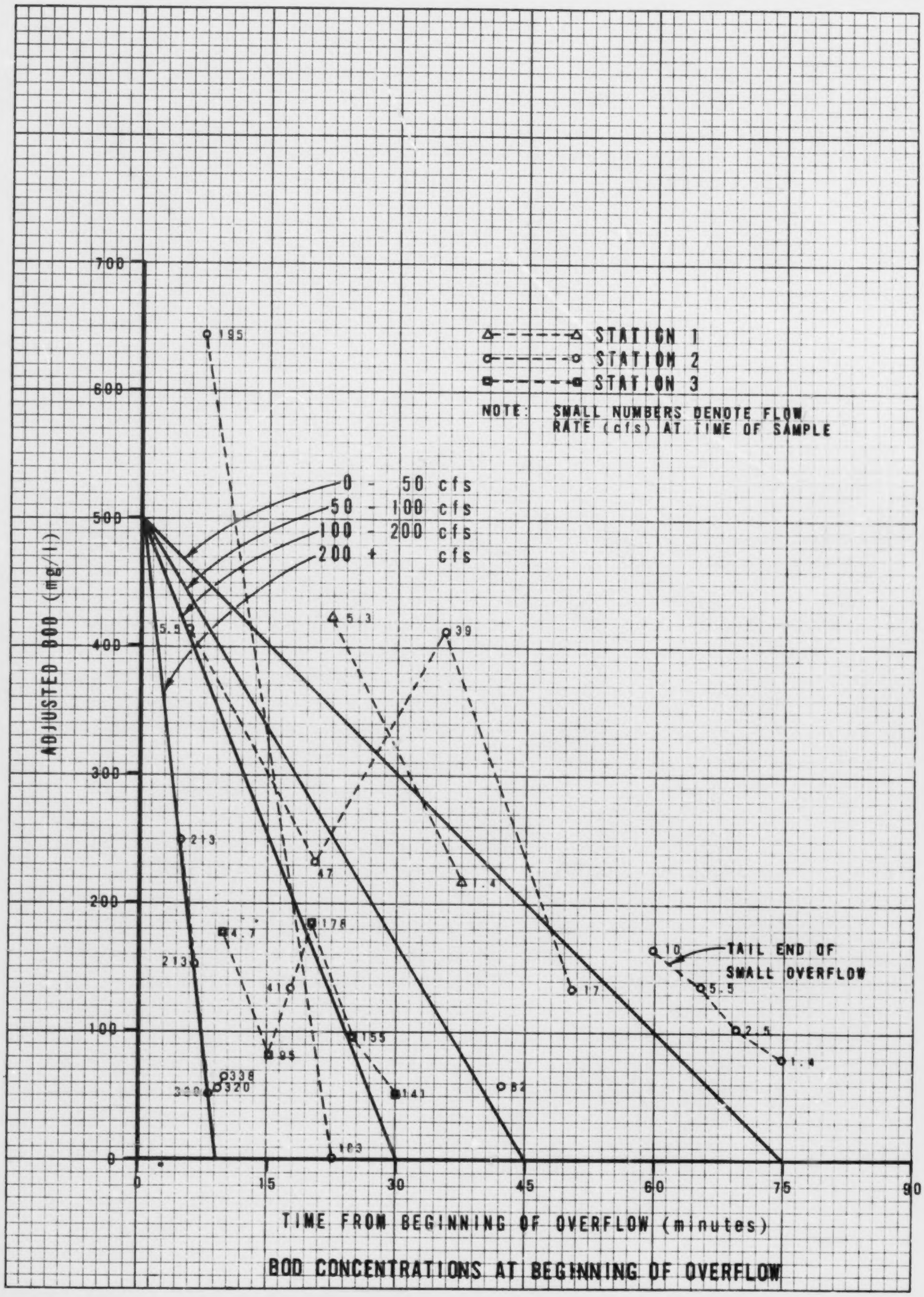
Boulevard: $\text{BOD} = 123.90 - 30.09 \log_{10} \text{FLOW (cfs)}$

McDaniel St: $\text{BOD} = 105.73 - 33.12 \log_{10} \text{FLOW (cfs)}$

(1) Destruction of a control weir at Confederate Avenue early in the study limited the reliability of subsequent high flow data at this station.

These equations permit estimation of the BOD component due to sanitary sewage entering the system during the overflow. Their statistical significance is quite strong, although the correlation coefficients, r, are not high. The resulting predicted concentrations are good estimates of average values occurring during the year, although specific storms may yield wide variations around these averages. Detailed analysis of the data at McDaniel Street indicated that adjustment of BOD concentrations, as described above, increased the variance accounted for at that station from 11.1 percent to 19.8 percent. Residual variance may be attributed to many factors, among which may be included the accuracy of the BOD test, accuracy of the rating curves at high flow rates, possible sampling bias, and length of the preceding dry weather period.

The components due to storm runoff and sanitary sewage, both of which are functions of flow rate, were subtracted from the record to leave a time–dependent residual at the beginning of each overflow. Insufficient usable data were available to analyze the relationship rigorously at each station. Consequently readings at all three stations were combined and linear regressions were fitted visually, as shown in Figure 24. Each line denotes an approximate trend evident in BOD concentrations in the flow range specified, the flow rates being shown beside each data point. For flows during the initial phase of less than 50 cfs, about 75 minutes are required to completely scour the combined sewer system and to remove all settled solids. By contrast, intense storms yielding initial flows greater than 200 cfs scour the system in approximately nine minutes. During this time BOD values are higher than would be possible from dilution of influent sewage alone. Peak concentrations of approximately 500 mg/l adjusted BOD occur at the beginning of overflows. Actual recorded values are higher, reflecting components due to storm runoff and influent sewage dilution. Concentrations decrease with time until the system is scoured, after which the latter two components predominate. Relatively high sewer slopes and low



BOD CONCENTRATIONS AT BEGINNING OF OVERFLOW

Figure 24

interceptor capacities are considered to cause these high BOD concentrations during the first few minutes of overflows. For a given flow rate, the decrease is assumed to be a linear function of time, although further data would clarify the nature of the relationship, whether linear or exponential. Table 6 shows the coefficients in the equation for each flow rate, equations being of the form:

$$\mathrm{BOD} = 500 - At \qquad t \leqslant B$$

TABLE 6

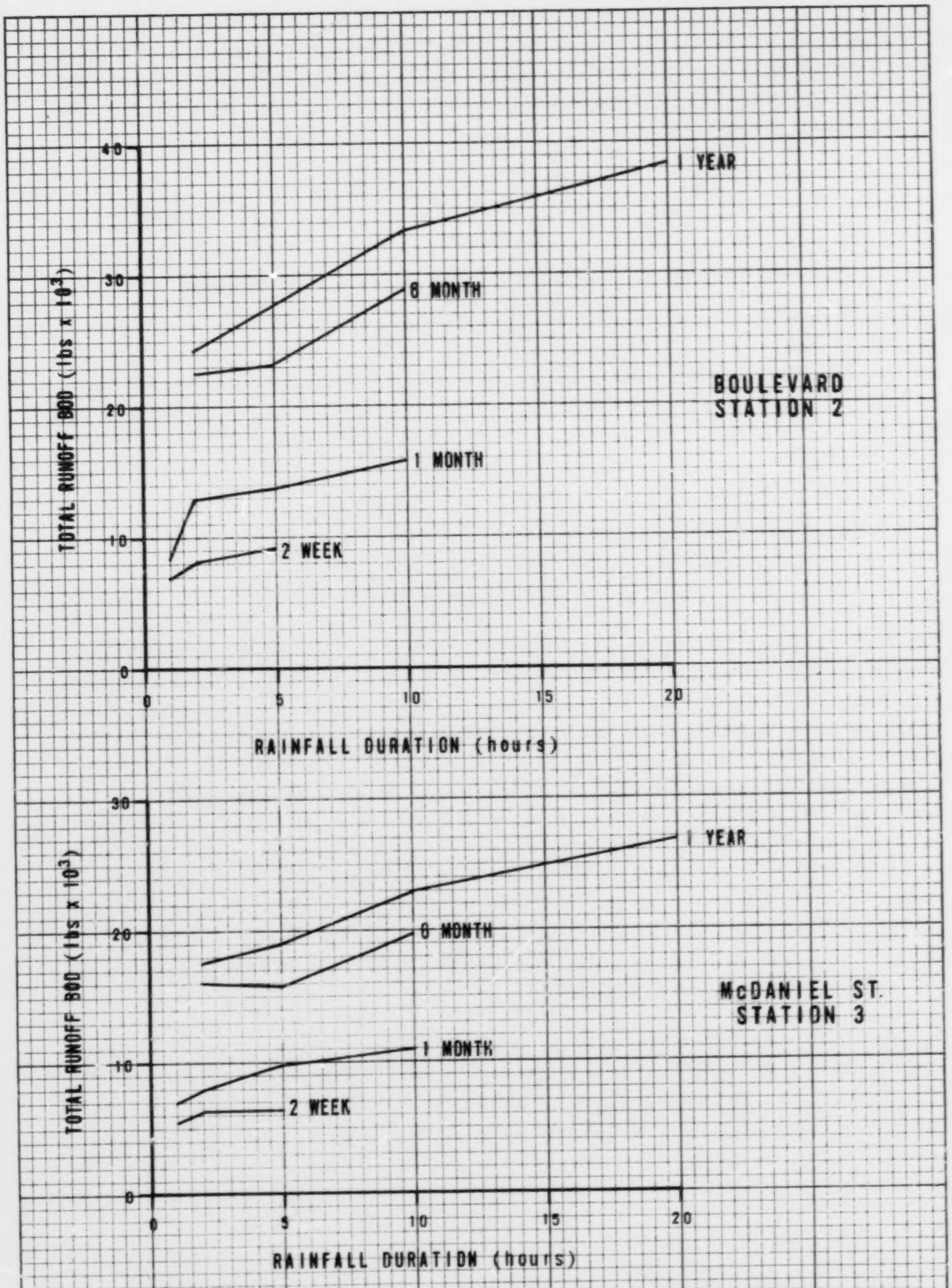
Coefficients for Various Flow Ranges – Initial BOD Equation

Flow Range (cfs)	A	B(Minutes)
0–50	6.67	75
50–100	11.10	45
100–200	16.67	30
200–over	55.60	9

Pollution Load From Overflows

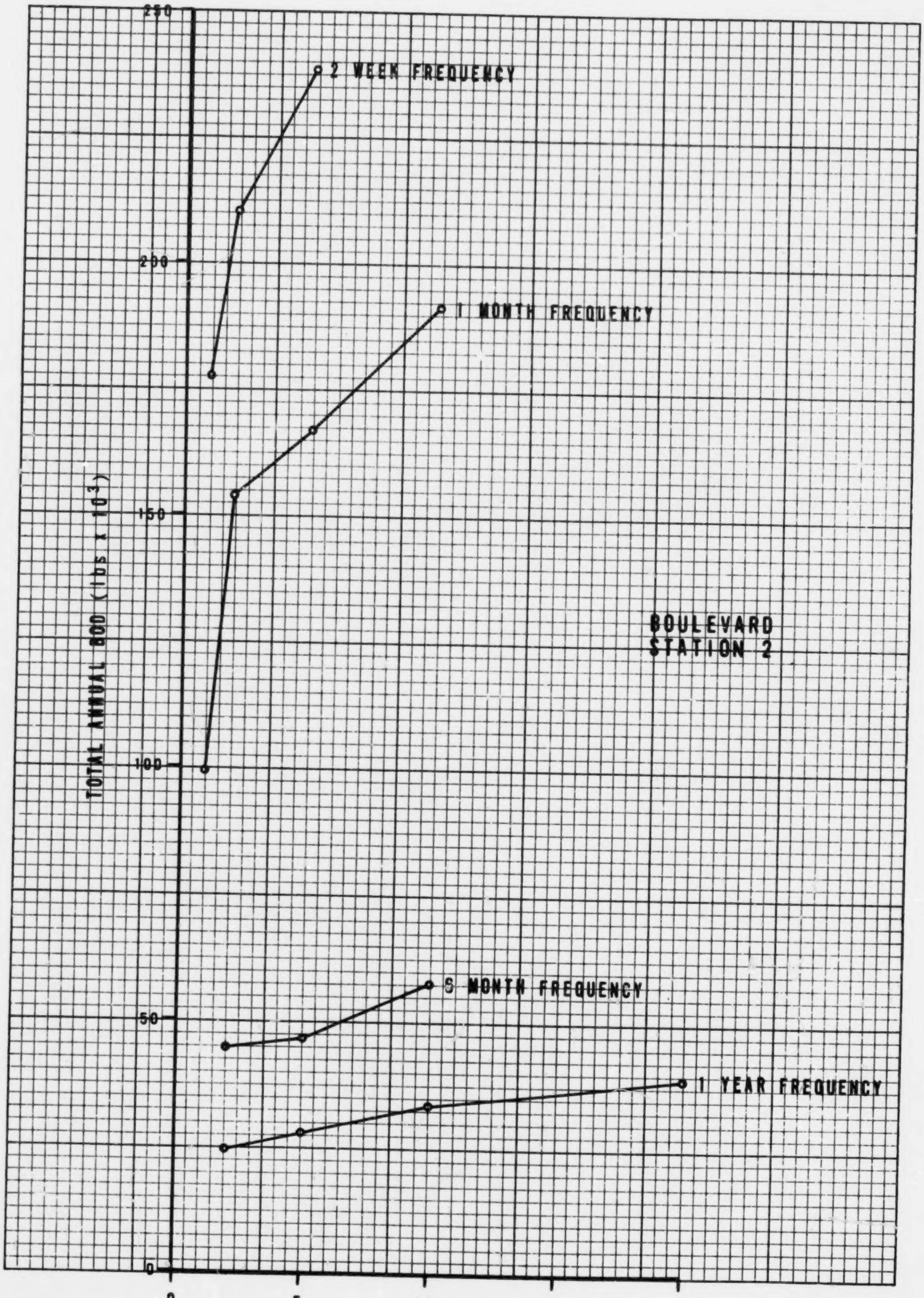
The BOD equations were incorporated into the runoff calculation computer program as a subroutine, as described previously in Section IV. This enabled estimation of concentrations and total pollution loads in each time increment of runoff resulting from application of each of the synthetic storms. Results for areas 2 and 3, Boulevard and McDaniel Streets, are shown in Figure 25. The relationship between total pounds of BOD and rainfall duration for storms of stated recurrence intervals indicates clearly the pollution severity of low frequency storms. However, when each curve is weighted by the number of expected occurrences per year, the great importance of high frequency storms becomes apparent, as shown for station 2, Boulevard, in Figure 26. These high frequency curves, especially the two– and four–week curves, should be considered conservative since they do not account for the cumulative nature of specific rainfall frequencies. The curves indicate that adequate treatment of small overflows from minor, frequent storms may significantly reduce the total annual pollutional load upon the South River.

Comparison of the results from Boulevard and McDaniel Street overflows is shown in Table 7 and is included in Figure 27. Values at each station were adjusted to pounds BOD per acre per hour of rainfall. Although discussion of these results is deferred until Section X, it may be observed that the values for each area are quite similar.



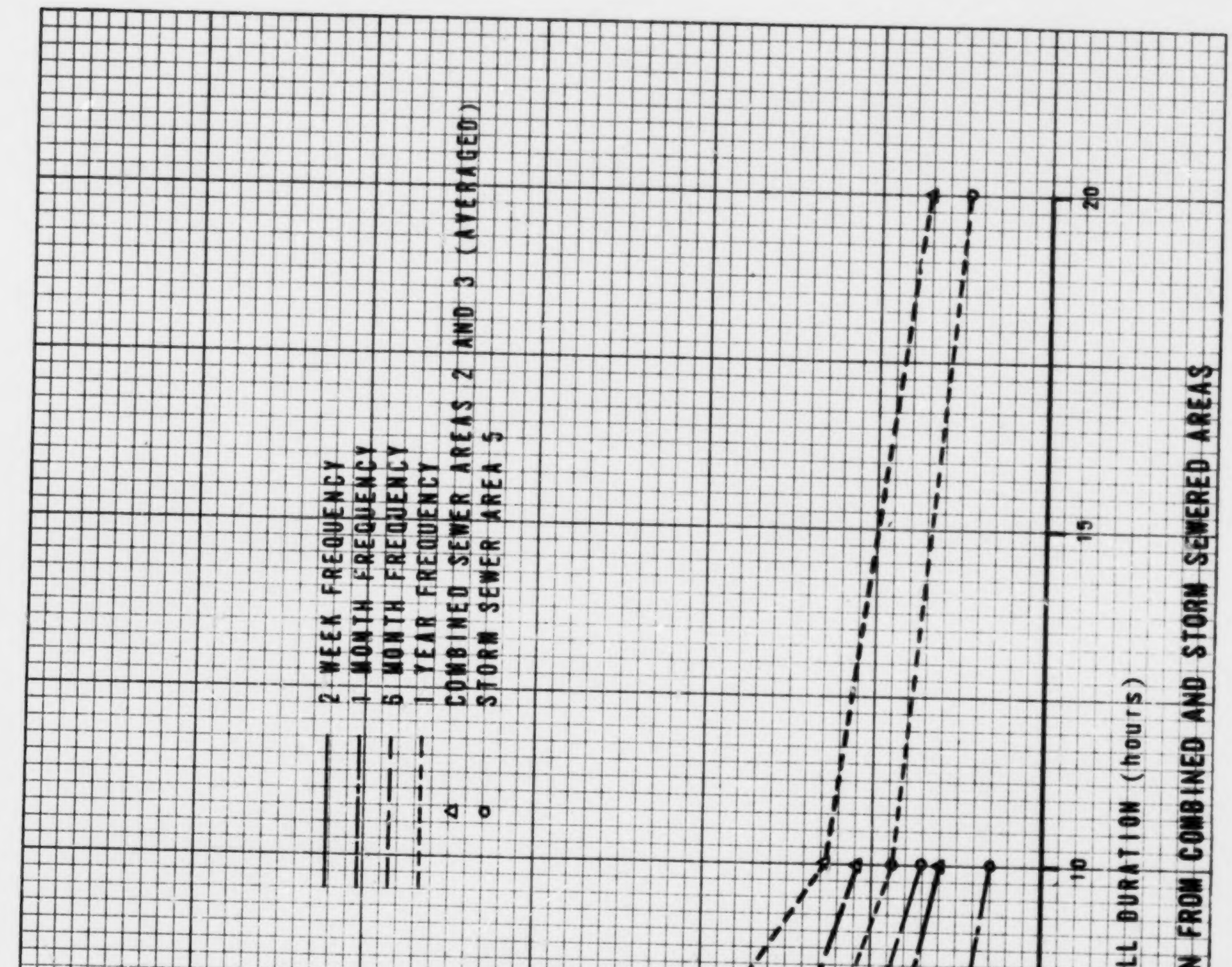
TOTAL RUNOFF BOD VS. RAINFALL DURATION FOR SINGLE, HIGH FREQUENCY STORMS

Figure 25



TOTAL ANNUAL BOD VS. RAINFALL DURATION FOR HIGH FREQUENCY STORMS

Figure 26



COMPARISON OF BOD POLLUTION FROM COMBINED AND STORM SEWERED AREAS

Figure 27

TABLE 7

Frequency	Duration (hours)	Pounds BOD/acre/hour of rainfall	
		Boulevard	McDaniel St
2 weeks	1	2.815	2.802
2 weeks	2	1.668	1.606
2 weeks	5	0.755	0.634
1 month	1	3.414	3.495
1 month	2	2.648	2.030
1 month	5	1.144	1.002
1 month	10	0.656	0.564
6 months	2	4.620	4.138
6 months	5	1.908	1.621
6 months	10	1.185	1.010
1 year	2	4.993	4.495
1 year	5	2.282	2.952
1 year	10	1.375	1.179
1 year	20	0.792	0.654

The figures of Table 7 were obtained from simulation of storms with peaks at one–quarter of the duration, and are considered to be reasonable estimates of combined sewer pollution in the study area. To determine the effect of rainfall distribution on the results, the series was rerun with the peaks at three–quarters of the duration. For both combined sewer areas the difference in total pounds of BOD was less than four percent, with the early runoff peak producing consistently higher results. It was concluded that for a given area the time distribution of rainfall during a typical storm of a given frequency and duration has little effect on the pollution load from that storm.

Slight irregularities in the smoothness of the curves in Figures 25 and 26 may be attributed partly to small sample error in the calculation of the rainfall intensity–duration–frequency curves of Figure 7. A longer record would have increased the definition of the curves, particularly at the extremes. However, the extra effort involved was not considered to be justified for the purposes of this investigation, since already well–defined high frequency storms are of primary interest in combined sewer studies.

Available treatment methods for combined sewer overflows, to be discussed later, include the storage and subsequent treatment of all, or part of, the runoff. For this reason the time variation of BOD loading at each overflow point was determined for storms having peaks at one–quarter of the duration. The relationship between cumulative runoff volume and percent of total BOD load is shown in Figures 28 and 29. The figures may also be interpreted as percent of overflow BOD stored in reservoirs of given volume for storms of varying frequency and duration characteristics.

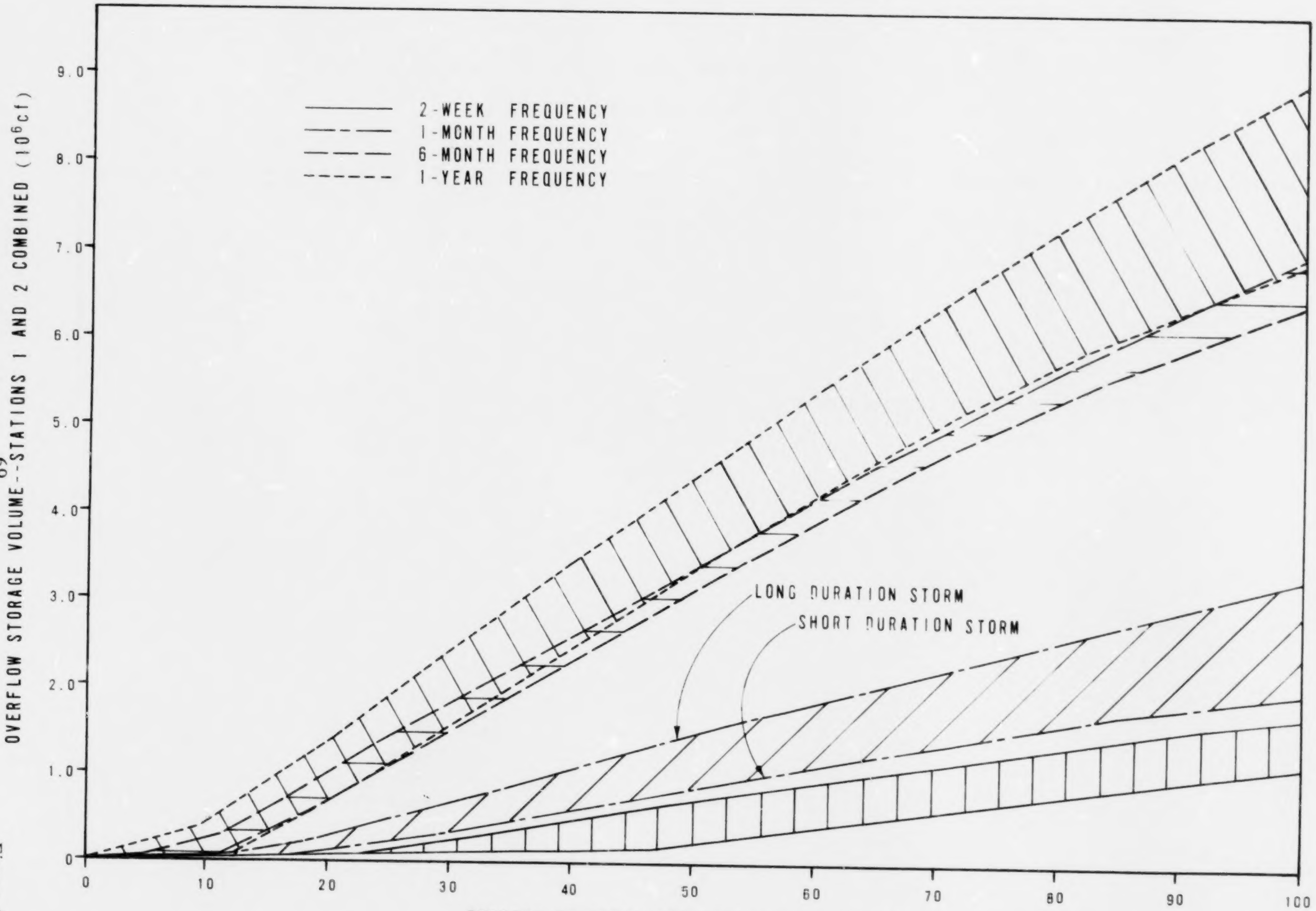
Figure 28 includes combined overflow from stations 1 and 2, Confederate Avenue and Boulevard. Both discharge into Intrenchment Creek, their point of confluence forming the site of a possible reservoir. Rainfall and runoff data for the subbasin contributing to the Confederate Avenue overflow were insufficient to define adequately the pollution characteristics of that area. Since the two areas are adjacent and land uses are very similar, total runoff and BOD were assumed to be directly proportional to basin area such that the combined runoff was 1.4 times the runoff from station 2 alone. During the storm the time distribution of BOD from stations 1 and 2 is assumed to be identical, although BOD from station 1 tends to be higher, due to the character of the industrial wastes entering the system in this area.

Results indicate that a reservoir at the Intrenchment Creek site storing 1.8 million cubic feet would contain 100 percent of the overflows from storms occurring with frequency greater than, or equal to, two weeks as well as 60 to 80 percent of BOD from one–month storms. Increasing the storage to 3.4 million cubic feet allows containment of overflow from all one–month storms as well as 40 to 50 percent of six–month and one–year storms. Similarly for the area contributing to station 3, the McDaniel Street overflow, storage of 0.5 million cubic feet will allow containment of 100 percent of the overflow BOD from all two–week storms as shown in Figure 29. It will also contain 55 to 90 percent of the overflow from all one–month storms and 25 to 40 percent of overflow BOD from one–year storms.

The significance of these results becomes apparent after consideration of Figure 10 discussed in Section IV. The approximate location on this graph of a two–week, a one–month, and a six–month storm was determined by averaging the locations found by two procedures. In the first procedure the two–week storm was located by finding that rainfall volume exceeded by 25 storms during the year. In the second procedure it was located by determining the volume of rainfall from the storm of average duration for the given frequency.

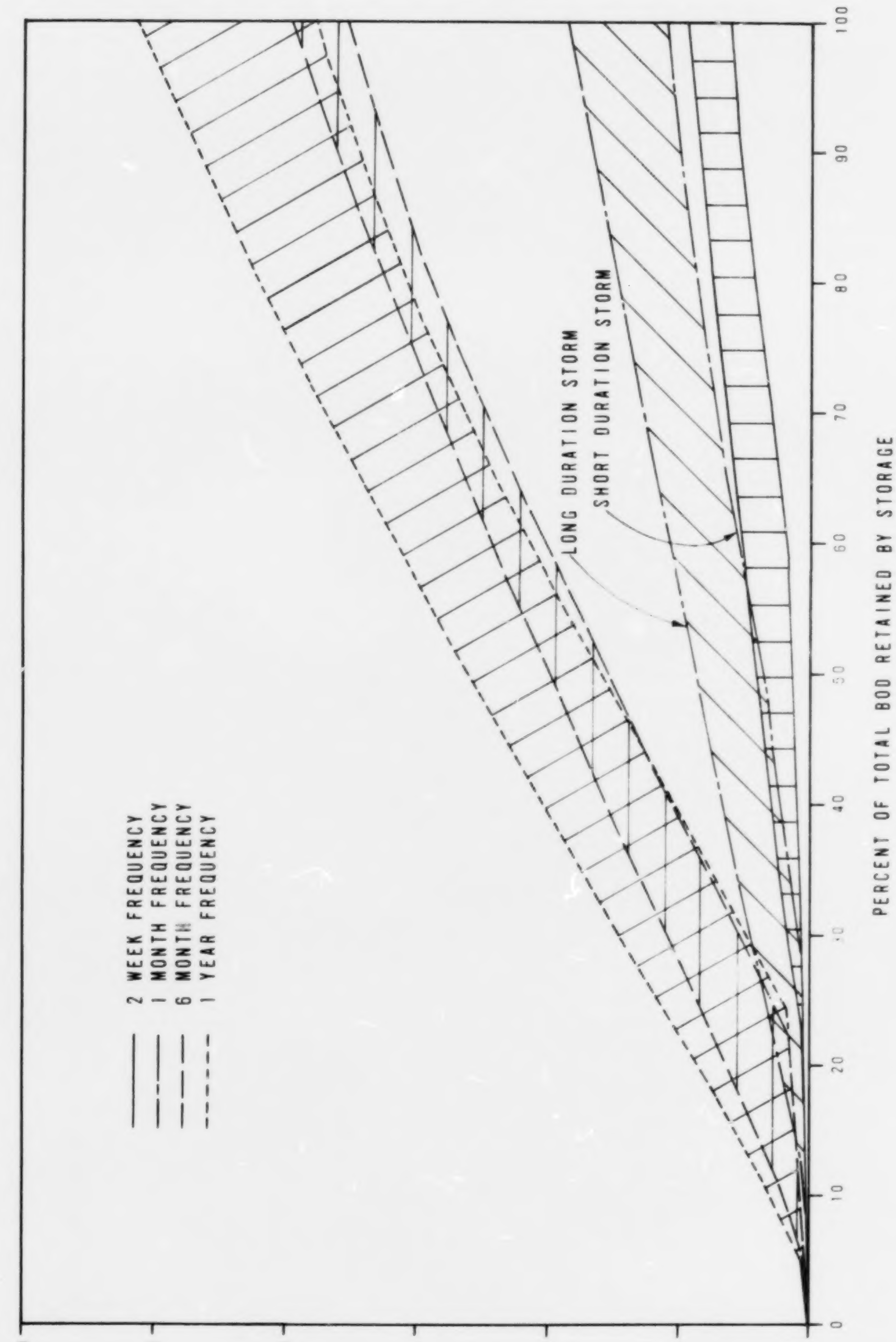
Consideration of a longer period of record would improve confidence in the location of the six–month storm in this figure. However, such precision on relatively low frequency storms was not necessary for this study. Of 118 storms occurring in 1969, approximately 57 percent were of higher frequency than two weeks. It may be concluded that a large percentage of the total annual BOD from combined sewers is due to storms with frequency greater than, or equal to, two weeks. Data for 1969 indicated that this is about 57 percent.

While total annual BOD is an important criterion of combined sewer pollution, the shock loading placed on receiving waters due to a single storm is of greater importance. The



RESERVOIR STORAGE OF BOD FROM BOULEVARD AND CONFEDERATE OVERFLOWS

Figure 28



RESERVOIR STORAGE OF BOD FROM McDANIEL ST. OVERFLOW

Figure 29

relationship of this shock loading to storm frequency is discussed in Section XI, CONDITIONS IN THE STREAM.

Consideration of Figures 7, 10, 27 and rainfall records for 1969 enabled calculation of the estimated total annual BOD from combined sewer overflows in Atlanta. The estimated total is 2,100,000 lbs BOD/year, amounting to 460 lbs BOD/acre/year. This value may be compared to results of a study by Engineering–Science (1967) of combined sewer pollution at Selby Street and Laguna Street in San Francisco. Values reported were 101 and 136 lbs BOD/acre/year respectively. Total annual rainfall for these areas is approximately one–quarter of that for Atlanta.

To this total should be added an additional 920,000 lbs/year bypassed at the treatment plants, of which 675,000 lbs/year are released from the Intrenchment Creek wastewater treatment plant. Approximately three–quarters of this bypassing is caused by quality deterioration of intercepted combined flows.

Reservoir storage of overflow from these storms would appear to be a feasible solution. Offsetting the apparent simplicity of such a solutuion, however, is the bypassing procedure presently followed by the sewage treatment plants in the study area. As discussed in Section VII in greater detail, many intercepted storm flows are bypassed to the receiving stream due to their low quality, particularly at the Intrenchment Creek sewage treatment plant. Although overflows are caused by rainfalls as low as 0.01 inch, a large percentage of the runoff from the high frequency storms is intercepted. Since small, frequent storms constitute the major source of combined sewer pollution, interception and subsequent bypassing of storm flows constitutes a significant restraint on the effectiveness of overflow storage and treatment.

Variation of Physical and Chemical Parameters

At Combined Sewer Overflows

Total Suspended Solids (TSS)

Total suspended solids were observed to vary exponentially with flow rate at all three overflow points. Statistical analysis substantiated the relationship

$$TSS = A\,(FLOW)^{B}$$

and yielded the values for A and B listed in Table 8.

TABLE 8

Station Number	Station Designation	A	B	r^2	No. of Storms
1	Confederate Ave	75.62	0.33075	0.27	33
2	Boulevard	19.82	0.56847	0.44	70
3	McDaniel St	48.86	0.78482	0.36	64

With peak flows occurring near the beginning of many recorded storms, it was not possible to determine conclusively the existence of initially high values for total suspended solids at the beginning of runoff events. Direct observation and some sample analyses indicate that an initial effect may occur. However, the relationships above were utilized in this study to predict total suspended solids loads from storms of different frequencies and durations in the study area.

Results indicate that TSS discharged to the South River from combined sewers increases for storms of lower frequencies. For any given frequency, TSS increases with decreasing duration. For a one–year storm of two–hour duration, predicted peak concentrations were 1,000 mg/l at Boulevard and 4,800 mg/l at McDaniel Street. Total amounts discharged from the Boulevard and McDaniel Street overflows are included in Table 9.

TABLE 9

Frequency	Duration (hours)	Total Pounds TSS/acre/hour of rainfall	
		Boulevard	McDaniel St
2 weeks	1	07.61	021.93
2 weeks	2	04.66	013.88
2 weeks	5	01.44	003.91
1 month	1	18.89	062.50
1 month	2	10.05	033.30
1 month	5	03.57	011.06
1 month	10	01.33	003.82
6 months	2	44.00	177.80
6 months	5	10.27	036.82
6 months	10	04.54	015.85
1 year	2	48.80	198.40
1 year	5	13.75	051.60
1 year	10	05.91	020.83
1 year	20	02.36	007.73

The consistently larger amounts of TSS at the McDaniel Street overflow, approximately three times amounts at the Boulevard overflow, are probably due to the lack of sufficient data at high flow rates incorporated in the regression analysis. Results for Boulevard, however, lie well within the range of data acquired during the study.

Volatile Suspended Solids (VSS)

The relationship between volatile suspended solids and total suspended solids is shown in Table 10 for all three combined sewer overflows. Results indicate that VSS is approximately 30–50 percent of TSS during overflow events at these three stations. No further underlying relationships were apparent in the data.

TABLE 10

Station Number	Station Designation	No. of Samples	Average (VSS/TSS)	Standard Deviation
1	Confederate Ave	53	0.43	0.16
2	Boulevard	68	0.32	0.16
3	McDaniel St	71	0.49	0.24

Conductivity

Conductivities during dry weather flow were monitored at the three interceptors. Values fluctuated during the day, reflecting directly the hourly variation in flow rate. Averages at Confederate Avenue, Boulevard and McDaniel Street were 500, 450, and 700 micromhos per centimeter (μmhos/cm) respectively, with fluctuations of less than 200 μmhos/cm.

Variation of conductivities during overflows is similar to the variation of BOD, although the range of values is slightly dampened. Initially high values decrease with time at a rate proportional to the flow rate, after which dilution of dry weather flow by the storm flow is the primary factor. Data were insufficient to determine accurately the initial response of conductivities during overflows. However, initial values appear to decrease from a maximum of approximately 500 μmhos/cm to steady–state values of approximately 100 to 300, depending upon flow rate. The relationship between conductivity (C) and flow rate for the Boulevard and McDaniel Street overflows is as follows:

Boulevard
(r=0.75)

$$C = 227.18 - 47.14 \log_{10} (\text{Flow})$$

McDaniel Street
(r=0.48)

$$C = 274.42 - 67.49 \log_{10} (\text{Flow})$$

These relationships are statistically significant at the 99.95+ percent level.

pH

Recorded values for pH at all three overflow points do not differ appreciably from neutral conditions. Confederate Avenue overflows were slightly on the basic side, with pH's ranging from 7.0 to 10.0. Values at Boulevard ranged from 6.0 to 8.0. At McDaniel Street recorded values ranged from 6.15 to 9.70, although the majority were between 6.5 and 8.0.

Acidity

Total acidities at the Confederate Avenue, Boulevard and McDaniel Street overflows averaged 5, 12, and 7 mg/l $CaCO_3$ respectively. Although the quantity of data was limited at the latter two monitoring stations, it is apparent that acidities are low. Further data would be required to determine whether discharge of acidic industrial wastes is insignificant or is well diluted by storm runoff.

Alkalinity

Alkalinity data at the three overflow points indicate that values at Confederate Avenue are generally highest. When combined with the slightly basic pH's found at this station, this implies that alkaline industrial waste discharges contribute to flows at Confederate Avenue, yielding an average alkalinity of 66 mg/l in overflows of less than five cfs. Average values of 30 and 34 mg/l were recorded at Boulevard and McDaniel Streets, respectively, with coefficients of variation ranging from 0.30 to 0.35 at all three stations.

Phosphate

Ortho phosphate concentrations at all three overflows showed no dependency upon time or flow rate. Occasional high concentrations suggest intermittent discharge to the sewer system of industrial wastes containing phosphate. Table 11 shows averages at the three stations, as well as standard deviations and recorded maximum concentrations.

TABLE 11

Station	No. of Samples	Average PO_4 (mg/l)	Standard Deviation	Maximum Concentration
Confederate Ave	67	6.5	5.1	25.6
Boulevard	81	1.7	1.3	06.2
McDaniel St	99	2.3	2.8	12.3

Of the three stations, Confederate Avenue yields the highest average and maximum orthophosphate concentrations. Total phosphates were not determined at the combined sewer monitoring stations.

Coliforms

Total coliforms, fecal coliforms and fecal streptococci were determined during the study at all monitoring stations except Confederate Avenue. Results at the remaining two combined sewer overflow points are summarized below in Table 12. Analyses were performed with the membrane filter technique.

TABLE 12

		Geometric Mean (MPN/ml x 10^2)		
Station	Number of Samples	Total Coliform	Fecal Coliform	Fecal Streptococci
Boulevard	21	1390	5	11
McDaniel Street	21	660	5	4

Chemical Oxygen Demand (COD)

Analysis of the data to determine points where COD increased and BOD decreased, thereby indicating toxic agents in the flow, was hampered by long sampling intervals, lack of sequential COD/BOD pairs and wide variability in flow rates between consecutive samples. An arbitrary maximum COD/BOD ratio of 5.0 was therefore selected, above which toxicity was assumed to be present. Ten pairs of data were rejected by this criterion. A summary of the remaining data for combined sewer overflows is shown in Table 13.

TABLE 13

				COD/BOD Ratio	
Station Number	Station Designation	No. of Samples	Avg COD	Average	Standard Deviation
1	Confederate Ave	47	442	2.11	0.73
2	Boulevard	47	164	1.95	1.24
3	McDaniel St	73	286	2.15	1.20
			297		
	Average			2.07	

Dissolved Oxygen (DO)

A small number of samples from the Boulevard and McDaniel Street overflows were tested for dissolved oxygen. Averages were 7.9 and 9.2 mg/l, respectively. Determination of the percent of saturation concentration for each sample was not possible. However, the average saturation concentration in the South River during 1968–1969 was 8.25 mg/l. It appears probable, therefore, that dissolved oxygen in combined sewer overflows is at saturation concentration. This indicates that insufficient flow time has elapsed for biochemical reaction to occur in the combined sewers. After discharge to the receiving stream, the saturation level of DO is not maintained.

SECTION VI

STORM WATER FLOWS

Sites Selected for Study

Three drainage areas within the upper South River basin were selected for study of storm water quantity and quality. These areas are slightly different in land use and other characteristics, but all are served by separate sanitary and storm sewer systems. Location of the areas and corresponding monitoring sites can be seen in Figure 30.

A summary of general information on the three areas appears in Table 14.

TABLE 14

SITES SELECTED FOR STORM WATER RUNOFF MONITORING

Station Number	Station Designation	Drainage Area, Acres	Estimated Population in Drainage Area	General Information
4	Harlan Drive	954	9,250	Open stream receiving runoff from industrial and residential areas
5	Casplan Street	517	3,750	Stream originating at storm sewers
6	Federal Prison Branch	1,498	7,250 (+ inmates)	Stream draining reserved land, also receives urban runoff

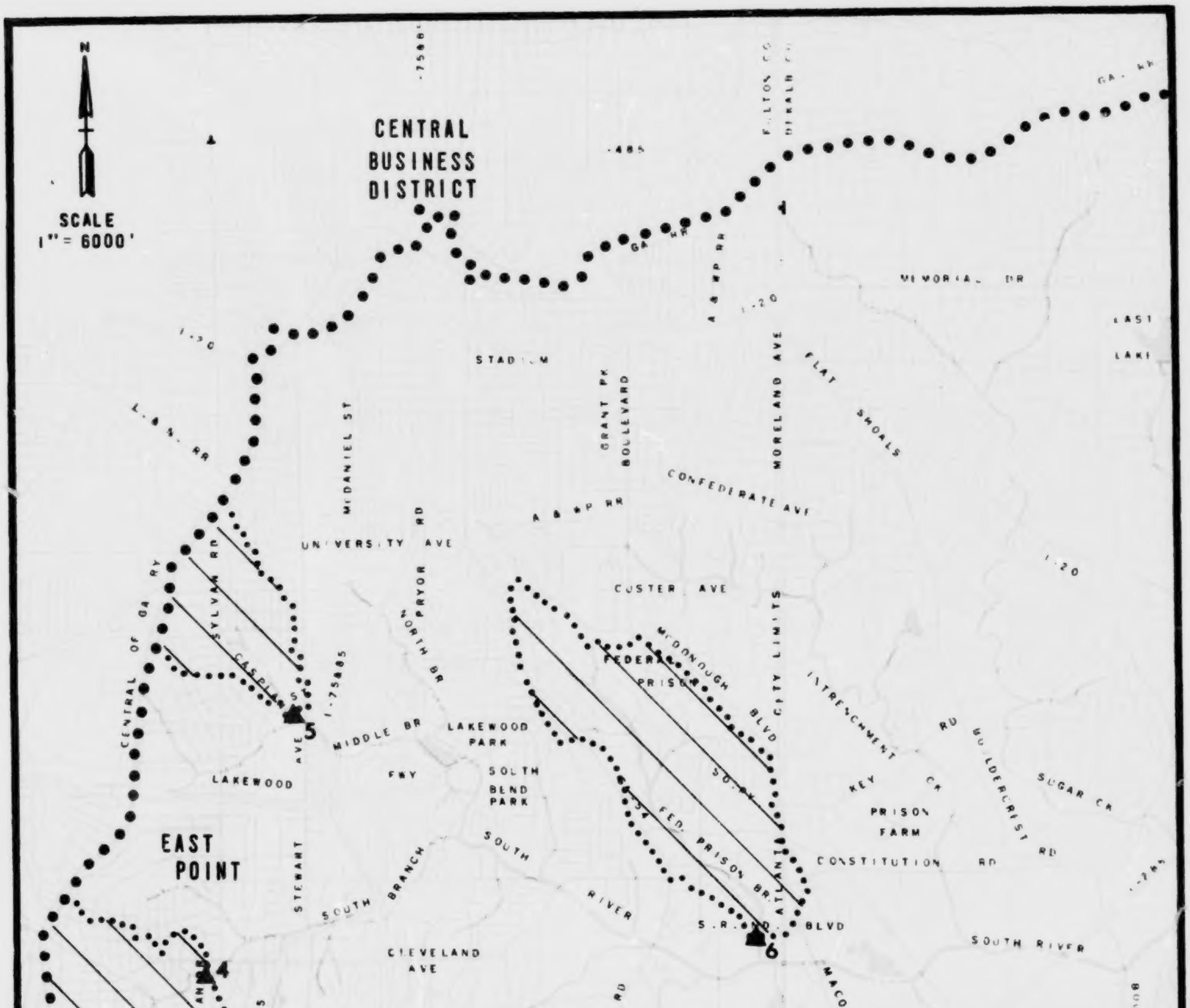

N
SCALE
1" = 6000'
CENTRAL BUSINESS DISTRICT
EAST POINT
HAPEVILLE
ATLANTA CITY LIMITS
MOUNTAIN VIEW
FOREST PARK
LAKEWOOD
LAKEWOOD PARK
SOUTH BEND PARK
STADIUM
FEDERAL PRISON
PRISON FARM
ATLANTA AIRPORT
ARMY DEPOT
CLAYTON CO
FULTON CO
DEKALB CO
MEMORIAL DR
FLAT SHOALS
CONFEDERATE AVE
UNIVERSITY AVE
CUSTER AVE
MCDONOUGH BLVD
CONSTITUTION RD
INTRENCHMENT CK
BOULDERCREST RD
SUGAR CK
KEY RD
SOUTH RIVER
SOUTH BRANCH
FED. PRISON BR
MACON DR
JONESBORO RD
POOLE CREEK
CLEVELAND AVE
MCDANIEL ST
NORTH BR
MIDDLE BR
PRYOR RD
STEWART AVE
SYLVAN RD
CASPLAN ST
HARLAN RD
MORELAND AVE
GRANT PK BOULEVARD
CENTRAL OF GA RY
C. OF GA RR
A & WP RR
GA RR
L & N RR
SOUTHERN RY
I-20
I-75
I-85
I-285
I-485
GA 54
4
5
6
▲ 4 HARLAN DRIVE
▲ 5 CASPLAN STREET
▲ 6 FED. PRISON BRANCH
······· DRAINAGE AREA BOUNDARY
● ● ● ● SOUTH RIVER BASIN BOUNDARY
UPPER SOUTH RIVER DRAINAGE BASIN
STORM WATER RUNOFF MONITORING SITES

Figure 30

Station 4 is located on the south branch of South River. It utilizes an existing U.S. Geological Survey installation established in October, 1963, as a continuous recording station for the river's headwaters. The level recorder is installed upstream of the culvert on Harlan Drive in the City of East Point, Georgia.

The river falls approximately 100 feet in a distance of 1.5 miles down to this station. It receives runoff from industrial and residential areas located mainly within the City of East Point, although about 12 percent of the drainage area falls within the corporate limits of Hapeville. The stream at this station is usually clear at dry weather flows, but pollution probably originating at chemical industries in the area was found to reach the stream sometimes, producing sudden increases in the conductivity of its waters. Peak discharges observed during the few years of record have exceeded 800 cubic feet per second (cfs) at this station. Flows of over 600 cfs were recorded four times during the present study.

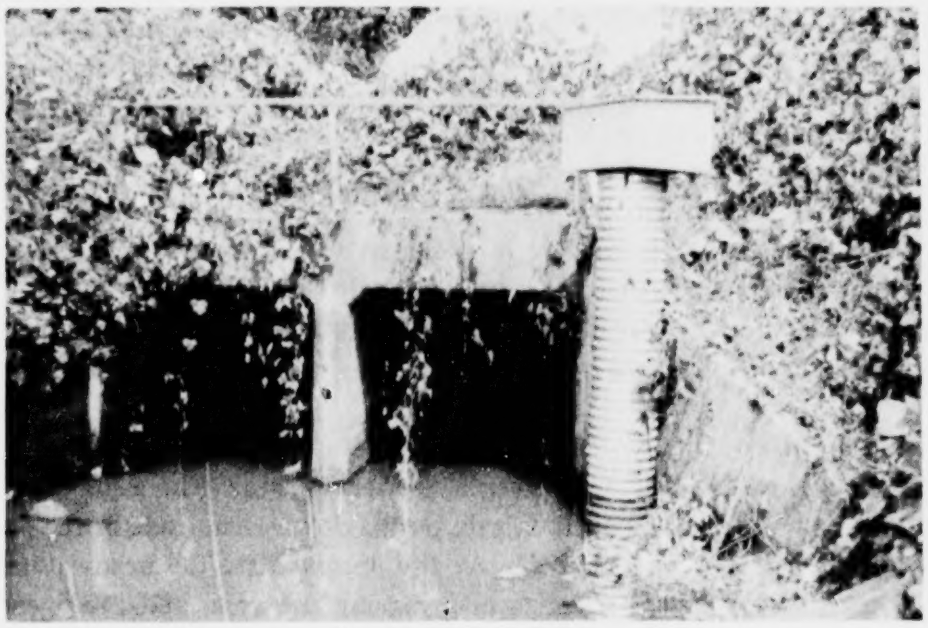
Station 5 is located on one of the two streams that form the middle branch of South River. These two streams merge just above the culvert at Stewart Avenue. The gaging station was established on the stream entering from the north, on the downstream side of the culvert at Casplan Street and can be seen in Figure 31. At the present time the headwaters of this stream consist entirely of the storm sewer system serving the area, and the open stream originates on the south side of Deckner Avenue at the end of a 6–foot arch brick storm sewer. A short distance downstream two 42–inch storm sewers contribute additional flows to the stream from the west side of the area.

The stream runs in the open for 3,400 feet through the Perkerson Park area, and falls 57 feet in this distance before reaching the culvert at Casplan Street, where the gaging and sampling station was established.

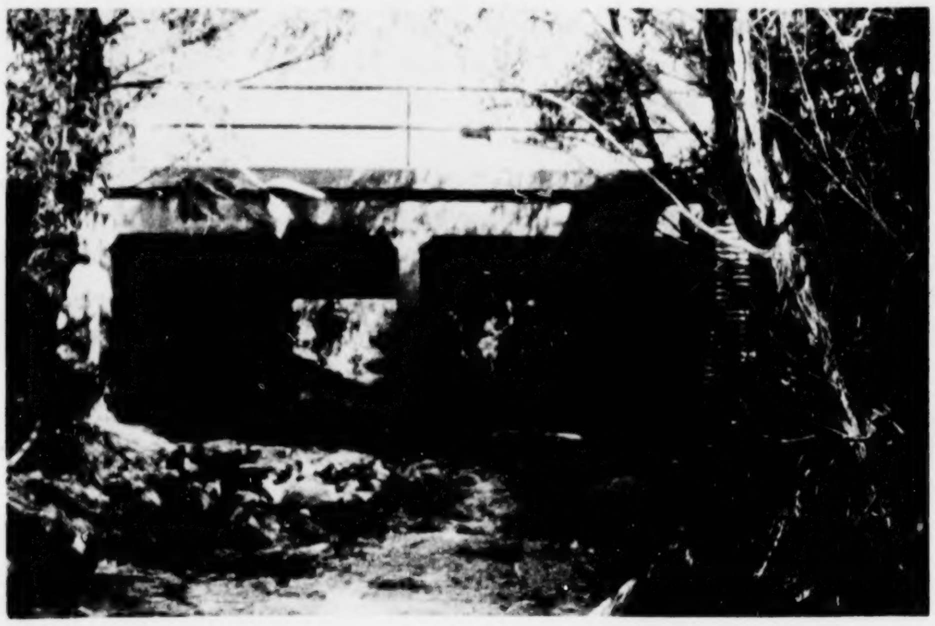
Station 6 is located on Federal Prison branch, a tributary entering South River from the north approximately 4,700 feet downstream of the Jonesboro Road bridge. The gaging station was installed downstream of the culvert on South River Industrial Boulevard, as shown in Figure 31.

This stream was selected mainly because, although its drainage area is still located entirely within the upper South River basin, land uses in part of it, at least, are rather different than for the preceding two streams. All of the reservation surrounding the prison, most of it sloping open land, falls within the drainage area. In contrast, most of the land in the other areas selected for study is highly developed.

The longest branch of this stream, flowing in a northwest–southeast direction, runs overland for a distance of 2.5 miles, from the end of the 6' x 6' arch culvert at Sawtell Avenue, before reaching the gaging station. A total fall of 140 feet takes place in this distance. Upstream of Sawtell Avenue most of the area is served by a separate sewer system. Storm sewers from this area end with a 72–inch diameter concrete pipe that discharges into the culvert at Sawtell Avenue.



Flow Level Recording Station at Casplan Street
Tributary to Middle Branch of South River.



Flow Level Recording Station at South River Industrial Boulevard
Federal Prison Branch

Figure 31

The drainage area of the Federal Prison branch is long and narrow, as opposed to the other two areas studied.

A small ponding area exists three–quarters of a mile downstream of Sawtell Avenue. A weir controls outflow from the pond, producing some flow regulation.

Federal Prison branch receives eventual discharges of raw sanitary sewage, when the Constitution Road pumping station fails to operate and overflows into the stream. Deterioration of the stream due to these discharges was noticed at least once during the study at the sampling station on South River Industrial Boulevard.

Flow Characteristics

Stations 4, 5, and 6 maintain small dry weather flows throughout the year, showing some seasonal variation. Approximate average annual values of these base flows are shown in Table 15, along with corresponding values at stations 7 and 8 on the South River at Jonesboro Road and Bouldercrest Road.

TABLE 15

Station Number	Contributory Area (square miles)	Approximate Base Flow (cfs)	Cubic Feet per Second per Square Mile
4	1.49	3.6	2.42
5	0.79	1.8	2.28
6	2.34	1.9	0.81
7	15.52	2.8	1.80
8	41.50	41.0	0.99

Stations 1, 2 and 3 are excluded from consideration since any base flow is combined with sewage and intercepted. Flows at station 8 represent the fraction remaining after removal of sewage treatment plant average annual flows.

Although the source is uncertain, average base flow is 1.66 cubic feet per second per square mile. Seasonal variation of the base flow implies that ground storage of rainfall occurs, although this is undoubtedly inhibited by the presence of clay and other impervious materials throughout the Atlanta area. The base flow may be attributed, therefore, to a combination of ground storage and flow from springs occurring in the area. Base flow at Harlan Drive is periodically augmented by industrial wastewater from cleaning operations.

BOD's of the base flow at stations 5 and 6 averaged 17 and 18 mg/1, respectively. Data were insufficient at station 4. Calculation of BOD's during storm flows was hampered by a lack of high flow rate observations at station 6. The relationship between BOD concentration and flow at station 5 was calculated as follows:

$$\text{BOD} = 29.2 + 8.28 \log_{10}\text{FLOW (cfs)} \qquad \begin{aligned} F &= 4.84 \\ r &= 0.287 \\ n &= 56 \end{aligned}$$

The regression equation is statistically valid, although the correlation coefficient is low. Although this equation was used in all computations, it is believed that further data at high flow rates would increase the value of the flow coefficient above 8.28.

Pollution Load from Storm Sewer Runoff

The subbasin contributing to station 5, Casplan Street, was selected for detailed study to determine pollution characteristics of a storm sewer area. Calculation of pollution concentrations in the runoff resulting from application of the synthetic storms, as described in Section IV, yielded the values listed in Table 16.

TABLE 16

Frequency	Duration (hours)	Pounds BOD/acre–hour of rainfall
2 weeks	1	1.149
2 weeks	2	0.770
2 weeks	5	0.334
1 month	1	2.187
1 month	2	1.320
1 month	5	0.634
1 month	10	0.318
6 months	2	3.664
6 months	5	1.279
6 months	10	0.730
1 year	2	3.940
1 year	5	1.580
1 year	10	0.892
1 year	20	0.465

Comparison of the above results with those listed in Table 7 for combined sewer overflows is shown in Figure 27, which shows that for storms of two– and four–week frequency, BOD loads from storm sewers are approximately half those of combined sewers. For less frequent storms, BOD loads approach three–quarters of those from combined sewers. The difference is minimized for short, intense storms. The annual average pollution from an acre served by storm sewers is estimated at 55 percent of that from a similar area served by combined sewers.

BOD loads from both storm shapes were calculated, confirming the observation of Section V that the time distribution of rainfall during a storm has little effect on the pollution load from that storm.

Variation of Physical and Chemical Parameters In Storm Sewers

Total Suspended Solids (TSS)

Acquisition of TSS data at Harlan Drive, Casplan Street, and the Federal Prison branch was limited, necessitating determination of weighted average concentrations for dry weather flow. This average was calculated to be 75 mg/l. Dry weather flow in the storm sewers was very small.

TSS concentrations during storm runoff at the three stations were combined and an exponential equation was fitted to the data. The equation

$$\mathrm{TSS} = 8.00\,(\mathrm{FLOW(cfs)})^{1.225}$$

indicates a sharper response of TSS to flow than for the combined sewer areas, probably due to the lack of significant base flow. Since this equation does not apply specifically to station 5, Casplan Street, no attempt was made to determine pounds TSS per acre per hour of rainfall at this station.

Volatile Suspended Solids (VSS)

The relationship between volatile suspended solids and total suspended solids is shown in Table 17 for all three storm sewer monitoring stations, and may be compared with values tabulated in Table 10 of Section V. Results indicate that VSS is approximately 20 to 40 percent of TSS during runoff events at these three stations. This is 10 percent less than for combined sewer overflows.

TABLE 17

Station Number	Station Designation	No. of Samples	Average (VSS/TSS)	Standard Deviation
4	Harlan Drive	8	0.40	0.16
5	Casplan Street	38	0.34	0.20
6	Federal Prison	21	0.21	0.14

Conductivity

Conductivities were recorded at station 4, Harlan Drive, for dry weather flow, yielding an average of 518 μmhos/cm with a standard deviation of 58. The high value is due to discharge of industrial wastes from Tennessee Corporation, Owens, Illinois, and other industries in the area. Lower conductivities were recorded at Casplan Street and Federal Prison branch, averaging 153 and 160 μmhos/cm, respectively, with low standard deviations. Both averages were for dry weather and low flows.

pH

The effect of industrial waste discharges upon pH at station 4, Harlan Drive, was apparent from the 4.6 to 6.3 range recorded during the study. Discharge of oxalic acid by the Tennessee Corporation may cause these low pH values. Neutral averages were recorded at Casplan Street and Federal Prison branch.

Acidity

Acidity at Harlan Drive averaged 111 mg/l $CaCO_3$, whereas values at Casplan and Federal Prison branch averaged 6 mg/l $CaCO_3$. When combined with conductivity and pH averages, the high value at Harlan substantiates conclusions concerning industrial pollution in the stream.

Alkalinity

Alkalinities at Harlan Drive, Casplan Street, and Federal Prison branch averaged 14, 80, and 43 mg/l, respectively. Occasional high values at Casplan Street reflected in a standard deviation of 80 mg/l indicate the possible intermittent discharge of alkaline industrial wastes in that area.

Phosphate

Average orthophosphate concentrations at the three storm sewer monitoring points are shown in Table 18. As in combined sewer overflows, standard deviations are high, although the averages are low. Total phosphates were not performed at these stations.

TABLE 18

Station	No. of Samples	Average PO_4 mg/l	Standard Deviation	Maximum Concentration mg/l
Harlan Dr	16	0.4	0.6	1.9
Casplan St	59	0.3	0.3	1.6
Federal Prison Branch	55	1.6	1.6	5.7

Coliforms

After analysis, coliform data proved inadequate to justify conclusions concerning coliform concentrations at storm sewer monitoring stations.

Chemical Oxygen Demand (COD)

COD/BOD ratios in all three storm sewers were higher than in combined sewer overflows. It was not possible to detect toxicity in the data, so all observations were included in the analysis, which is summarized in Table 19. The majority of observations were at dry weather or low flows.

TABLE 19

Station	No. of Samples	Average COD	COD/BOD Ratio	
			Average	Standard Deviation
Harlan Dr	13	28	4.00	1.8
Casplan St	38	84	4.20	2.9
Federal Prison Branch	27	67	2.60	1.5
Average		**60**	**3.60**	

Dissolved Oxygen (DO)

Averages of the values recorded at Harlan Drive, Casplan Street, and Federal Prison branch were 6.6, 9.0, and 5.6 mg/l, respectively. The samples from which these averages were obtained were small, thereby limiting reliability of conclusions concerning dissolved oxygen at storm sewer monitoring stations. However, these values are generally less than the annual average saturation concentration of 8.25 mg/1.

Comparison of Pollution Indices for Storm and Combined Sewer Areas

Table 20 summarizes physical, chemical and bacteriological analyses of storm flows from areas served by separate and combined sewers.

TABLE 20

Water Quality Parameter	AVERAGE CONCENTRATION					
	Combined Sewer Area			Storm Sewer Area		
	1	2	3	4	5	6
COD/BOD Ratio	2.11	1.95	2.15	4.00	4.20	2.60
VSS/TSS Ratio	0.43	0.32	0.49	0.40	0.34	0.21
Conductivity (μmhos/cm)	500	450	700	518	153	160
pH	8.5	7.0	7.2	5.4	7.0	7.0
Acidity (mg/l $CaCO_3$)	5	12	7	111	6	6
Alkalinity (mg/l$CaCO_3$)	66	30	34	14	80	43
Orthophosphate (mg/l)	6.5	1.7	2.3	0.4	0.3	1.6
Dissolved Oxygen (mg/l)	--	7.9	9.2	6.6	9.0	5.6
Total Coliform (MPN/ml x 10^2)	--	1,390	660	--	--	--
Fecal Coliform (MPN/ml x 10^2)	--	5	5	--	--	--
Fecal Streptococci (MPN/ml x 10^2)	--	11	4	--	2	--

SECTION VII

WASTEWATER TREATMENT PLANTS

Normal Operation

The City of Atlanta owns and operates two wastewater treatment plants within the South River basin. The South River plant is made up of two adjacent treatment facilities operating in parallel, one employing trickling filters and the other diffused air for secondary treatment. Intrenchment Creek plant, owned also by the City, is a high rate trickling filtration facility. Both plants receive sewage contributions from areas served by combined sewers, as well as from areas having separate storm and sanitary systems. Normal operation of these plants is based on treatment of dry weather flows only. Wet weather flows are generally bypassed to prevent overloading of the units and difficulties with the large amounts of grit that would otherwise reach the digesters.

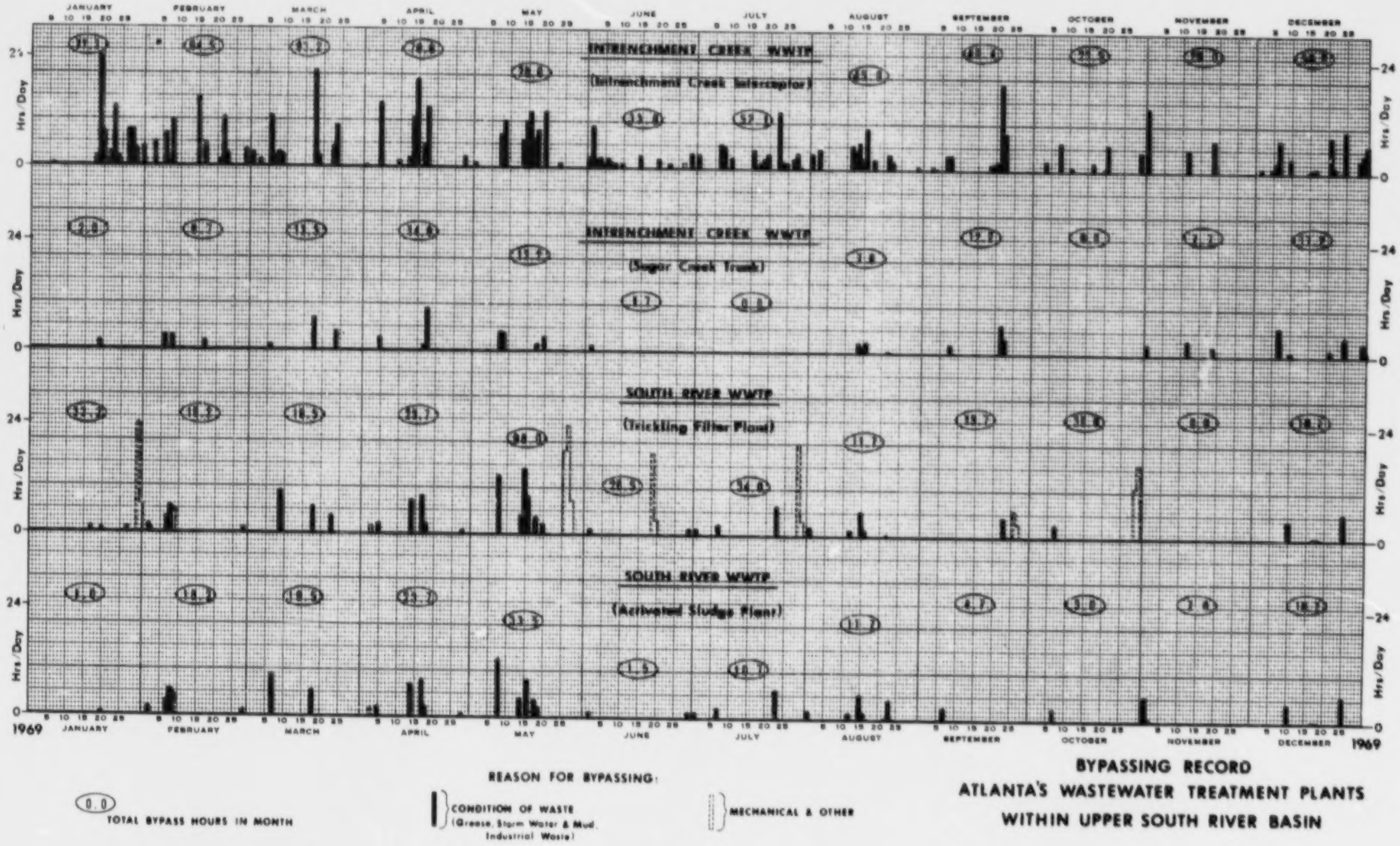
The fraction of combined sewer flows diverted to the treatment plants at interception points is augmented by additional sewage from separate systems before reaching its destination. Substantial increases in flow along the interceptor lines occur during storms. However, the primary cause of bypassing at the treatment plants is not the flow increase, but the poor quality.

The intercepted combined flows contain grease, grit and mud. Bypassing of all flows in the intercepted lines is necessary in order to prevent the grit from reaching the digesters. Grit enters the system from a number of sources, primary among which is the bypassing procedure utilized to separate mud from water treatment plant influent. This practice has recently been curtailed. Other sources include application of light gravel to highways during infrequent snowfalls, and construction activity within the City.

Due to the additional sewage flow carried by the interceptors, bypassing of intercepted flows at the wastewater treatment plants is more significant than discharge of the same flows at the interception points. Understanding of the fundamental difference between the two is important for understanding the remainder of the report.

The Intrenchment Creek treatment plant receives its inflow from two primary trunk sewers. One of these, the Intrenchment Creek interceptor, serves 46% of the area. It carries flow from the Boulevard and Confederate Avenue interceptors augmented by sewage from an additional 20% of the area served by the treatment plant. As shown in Figure 32, this interceptor is bypassed very frequently, averaging 52 hours per month during 1969. Condition of the waste was the reason for almost every bypass event. Simultaneous bypassing of flows from both the Intrenchment Creek interceptor and the Sugar Creek trunk sewer occurred an average of eight hours per month during 1969. The Sugar Creek trunk sewer does not carry flows from combined sewer areas.

TOTAL HOURS WHEN WASTEWATER TREATMENT PLANTS WERE BYPASSED



BYPASSING RECORD
ATLANTA'S WASTEWATER TREATMENT PLANTS
WITHIN UPPER SOUTH RIVER BASIN

Figure 32

Bypassing at the South River wastewater treatment plant occurs less frequently, as shown in Figure 32. The sewer carrying intercepted flow receives additional sewage from a large fraction of the area served by the treatment plant. Therefore, the impact of each bypass event is more severe, due to the higher fraction of sewage in the influent.

South River Wastewater Treatment Plant

The South River plant is located on the north side and almost within the flood plain of the river, just downstream of Jonesboro Road (State Road 54). As stated before, it actually consists of two adjacent plants operating in parallel, the older one utilizing standard rate trickling filters for secondary treatment, and the more recent one employing a modified activated sludge process. Sludge treatment for both plants is provided by a single group of units.

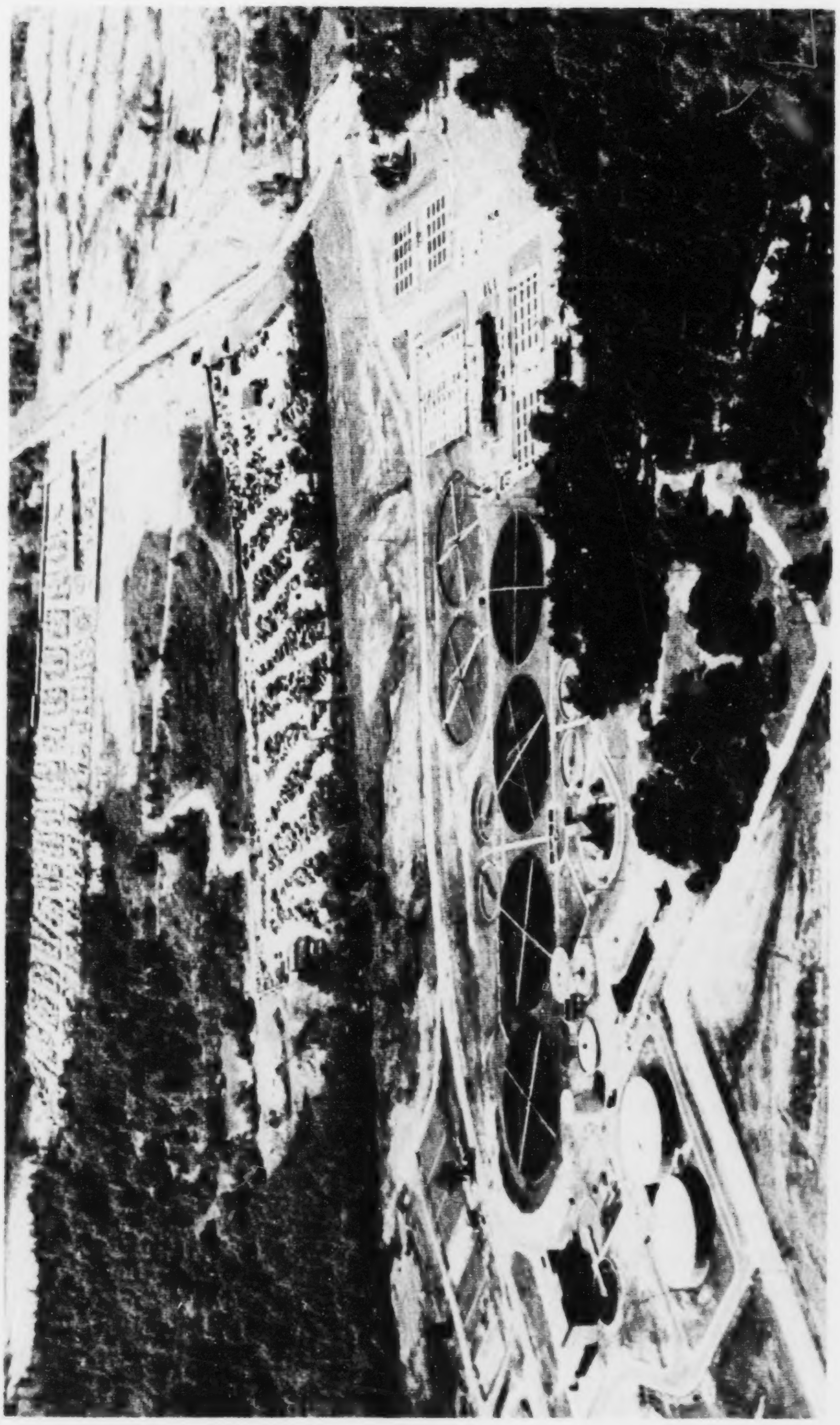
An aerial view of the plant can be seen in Figure 33, while flow diagrams of the existing treatment facilities appear in Figure 34.

The South River plant receives for treatment all sanitary sewage from most of the upper south River drainage basin upstream of the Jonesboro Road (State Road 54) bridge, plus a few smaller additional areas. A large part of the potential contributory area is completely sewered at the present time, and altogether encompasses 26.09 square miles, with an estimated population of 99,750. Additional areas contributing sewage include most of the Federal Prison branch basin, a tributary entering the South River downstream of the plant. Sanitary sewage from part of this area flows by gravity into the South River plant, while part is pumped at the Constitution Road pumping station into the gravity system.

Most of the contributory area is served by a separate sewer system, combined sewers being confined to approximately 2.07 square miles at the northwest end of the basin. This figure is equivalent to only 7.9 percent of the total contributory area. Part of the sewers carrying sanitary waste into the South River plant originate in areas within the cities of East Point and Hapeville to the west, as well as in Clayton County to the south. Potential contributory areas also extend into DeKalb County southeast of the plant.

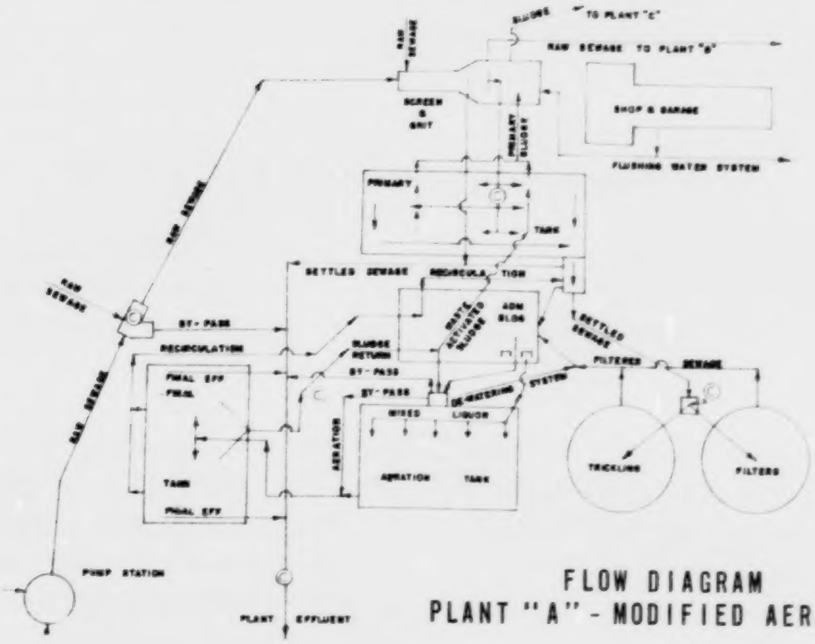
The trickling filter plant is designed to treat an average flow of 6.0 mgd, while the modified activated sludge plant has a design capacity of 12.0 mgd. Combined capacity of both is 18.0 mgd. An operating summary of these treatment facilities appears in Appendix C. Based on these figures, for the twelve–month interval December, 1968 through November, 1969, average daily flows entering both plants were 15.41 mgd, influent BOD's averaging 232 mg/l. Effluent BOD's averaged 40 mg/l for this period and BOD discharged daily into the stream was approximately 5,740 pounds.

From operating figures it appears that these two plants function at a rather high efficiency and that bypassing is not frequent. However, total pollutional loads in the effluent are high, annual mass discharge representing a large fraction of the total pollution entering the stream.

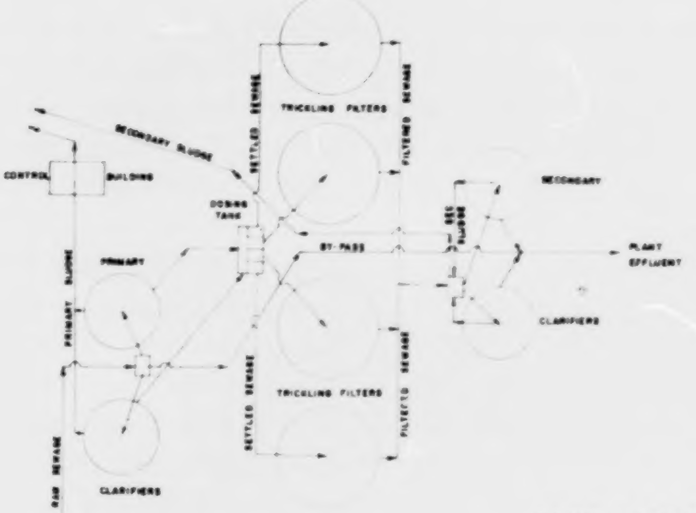


South River Wastewater Treatment Plant
City of Atlanta

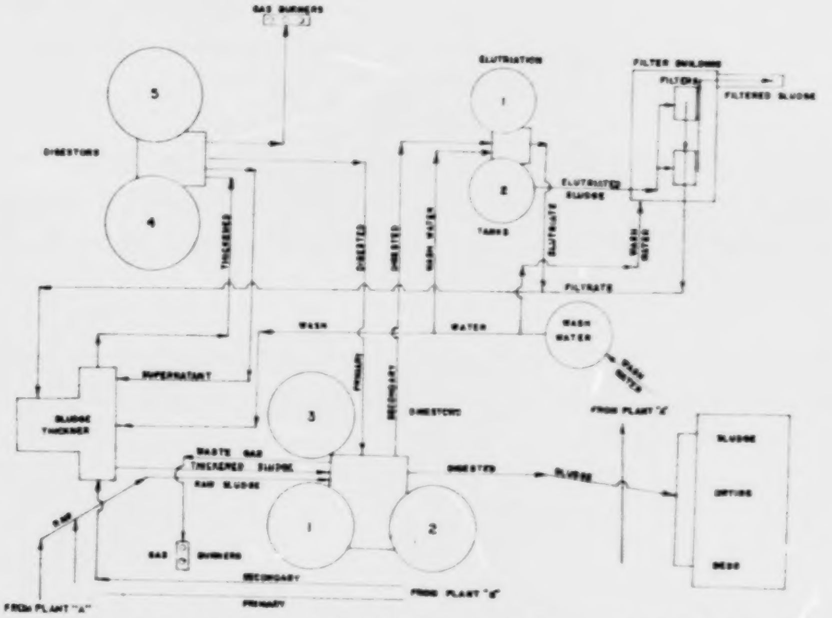
Figure 33


SLUDGE
TO PLANT "C"
RAW SEWAGE
RAW SEWAGE TO PLANT "B"
SCREEN & GRIT
PRIMARY SLUDGE
SHOP & GARAGE
FLUSHING WATER SYSTEM
PRIMARY
TANK
RAW SEWAGE
SETTLED SEWAGE
RECIRCULATION
ADM BLDG
WASTE ACTIVATED SLUDGE
SETTLED SEWAGE
RAW SEWAGE
BY-PASS
RECIRCULATION
SLUDGE RETURN
BY-PASS
BY-PASS
DE-WATERING SYSTEM
FILTERED SEWAGE
FINAL EFF
FINAL
TANK
FINAL EFF
AERATION
MIXED LIQUOR
AERATION TANK
TRICKLING
FILTERS
RAW SEWAGE
PUMP STATION
PLANT EFFLUENT

FLOW DIAGRAM
PLANT "A" - MODIFIED AERATION


TRICKLING FILTERS
SETTLED SEWAGE
FILTERED SEWAGE
SECONDARY SLUDGE
CONTROL BUILDING
DOSING TANK
SECONDARY
SEC SLUDGE
BY-PASS
PLANT EFFLUENT
PRIMARY
PRIMARY SLUDGE
CLARIFIERS
TRICKLING FILTERS
SETTLED SEWAGE
FILTERED SEWAGE
RAW SEWAGE
CLARIFIERS

FLOW DIAGRAM
PLANT "B" - TRICKLING FILTRATION


GAS BURNERS
5
4
DIGESTORS
ELUTRIATION
1
2
TANKS
FILTER BUILDING
FILTERS
FILTERED SLUDGE
ELUTRIATED SLUDGE
THICKENED
DIGESTED
DIGESTED
WASH WATER
ELUTRIATE
WASH WATER
FILTRATE
WASH
WATER
WASH WATER
WASH WATER
SUPERNATANT
SLUDGE THICKENER
3
PRIMARY
SECONDARY
DIGESTORS
FROM PLANT "A"
WASTE GAS
THICKENED SLUDGE
RAW SLUDGE
DIGESTED SLUDGE
SLUDGE
DRYING
BEDS
1
2
RAW
GAS BURNERS
SECONDARY
FROM PLANT "B"
PRIMARY
FROM PLANT "A"

FLOW DIAGRAM - SLUDGE TREATMENT FACILITIES

FLOW DIAGRAMS

SOUTH RIVER
WASTEWATER
TREATMENT PLANT

CITY OF ATLANTA

Figure 34

Average daily flows from the plant are as much as 10 times larger than the base flow in the stream at the point of discharge. On the other hand, during large floods the stream may carry as much as 100 times the average flow discharged by the plant.

Intrenchment Creek Wastewater Treatment Plant

This plant is located on a site adjacent to the creek just upstream of the Key Road bridge, in DeKalb County. The location is approximately 2.1 miles downstream of the city limits, measured along the stream, and 1.5 miles above the point where Intrenchment Creek enters South River.

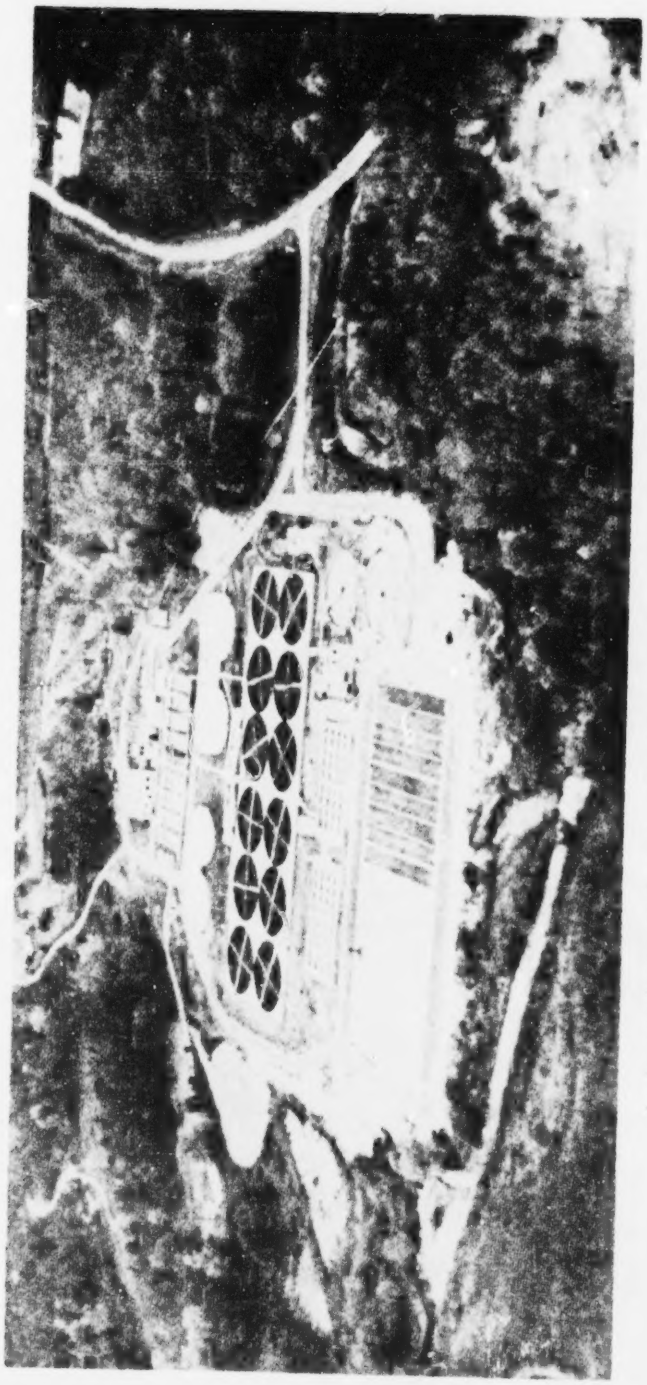
The plant presently receives sanitary sewage from an area of approximately 20.15 square miles, with an estimated population of 115,750. About 37.8 percent of this area, on the east side of the basin, is located in DeKalb County and outside Atlanta's city limits. Part of the upper Sugar Creek basin is included in the contributory area, sewage reaching the plant through the Sugar Creek trunk that cuts through the ridge separating this basin from the adjacent Intrenchment Creek.

On the northwest, the area contributing sanitary sewage to the Intrenchment Creek plant is served by combined sewer systems, including the Boulevard (Lloyd Street) and Confederate Avenue (Stockade) trunks. These two systems encompass a majority of the combined sewers within the South River basin. Altogether, they serve approximately 5.30 square miles, representing 26.2 percent of the total area served by Intrenchment Creek plant.

The plant has a design capacity of 20.0 mgd and provides secondary treatment by means of high rate trickling filters. Figure 35 is an aerial view of the plant, and Figure 36 is a flow diagram. An operatingsummary for this plant during the interval December, 1968, through November, 1969, appears in Appendix C. From available figures, average daily flows entering the plant during this period were 13.29 mgd, with an average influent BOD of 368 mg/l. Effluent BOD's averaged 62.8 mg/l, and pollutional loads discharged into the stream represent 8,800 pounds of BOD per day. The plant operates at a rather high efficiency for the type of process employed. However, effluent BOD is very high, and need of bypassing is rather frequent, mainly due to quality of the sewage in the Intrenchment Creek interceptor. The combined sewer systems cover 56.6 percent of the area served by this interceptor. Need for bypassing sewage from Sugar Creek trunk is not frequent, and storm water is reported to reach this trunk only during very heavy rains.

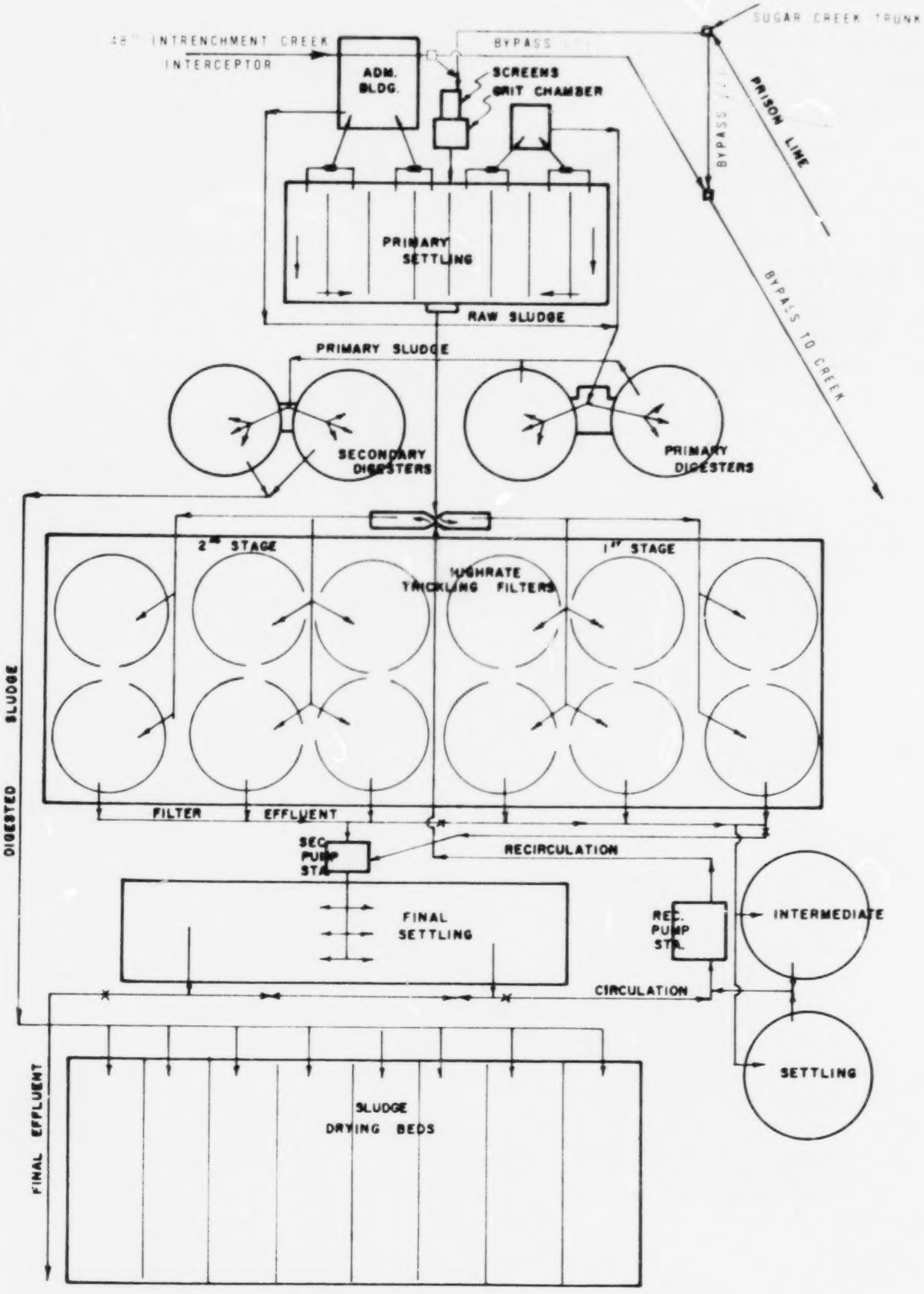
Intrenchment Creek, which receives the effluent and bypasses from this plant, originates at the two major combined sewer overflow points within the basin. Drainage area down to the plant is only 9.9 square miles, average daily effluent volumes from the plant exceeding dry weather flows on the creek at this point.

At the confluence point, dry weather flows in both Intrenchment Creek and South River consist chiefly of wastewater treatment plant effluents. Control Station No. 8 of the study



Intrenchment Creek Wastewater Treatment Plant
City of Atlanta

Figure 35


48" INTRENCHMENT CREEK
INTERCEPTOR
ADM. BLDG.
BYPASS
SUGAR CREEK TRUNK
SCREENS
GRIT CHAMBER
PRISON LINE
BYPASS
PRIMARY SETTLING
RAW SLUDGE
BYPASS TO CREEK
PRIMARY SLUDGE
SECONDARY DIGESTERS
PRIMARY DIGESTERS
2nd STAGE
1st STAGE
HIGHRATE TRICKLING FILTERS
DIGESTED SLUDGE
FILTER EFFLUENT
SEC. PUMP STA.
RECIRCULATION
FINAL SETTLING
REC. PUMP STA.
INTERMEDIATE
CIRCULATION
SETTLING
SLUDGE DRYING BEDS
FINAL EFFLUENT

FLOW DIAGRAM

INTRENCHMENT CREEK
WASTEWATER TREATMENT PLANT

CITY OF ATLANTA

Figure 36

is located 1.1 miles below this point, at the Bouldercrest Road bridge. At this station conditions in the river are most critical. Dry weather flow records at Bouldercrest clearly reflect hourly flow variations in plant effluents.

Other Treatment Facilities

A few more wastewater treatment plants contribute additional pollution to South River between Bouldercrest and the Klondike Road bridge. DeKalb County owns and operates two secondary treatment plants within the basin. The Shoal Creek plant, located upstream of the I–285 bridge at this stream, is a standard rate trickling filter facility with a design capacity of 2.0 mgd. Flows averaged 2.62 mgd during 1969, with an estimated BOD discharge of 962 pounds per day to the stream. The Snapfinger Creek plant, located on the north side of South River just below the Flakes Mill Road bridge, is a high rate trickling filter facility with a design capacity of 3.0 mgd. It is presently being enlarged to a capacity of 6.0 mgd. Average flows received during 1969 were 2.49 mgd, with estimated daily discharge of 1,095 pounds BOD to the South River. A summary of operating data for these two plants is included in Appendix C. Within the Snapfinger Creek basin is a waste treatment lagoon serving part of Stone Mountain. Effluent enters Barbashela Creek, a tributary within the basin.

Long range plans for the Metropolitan Atlanta area consider future abandonment of Shoal Creek and other existing treatment plants upstream, and gravity conveyance of wastes from all the upper basin and the Stone Mountain area into the present Snapfinger Creek plant location, for treatment at a larger, more efficient facility.

The Panola industrial area, located within the Pole Bridge Creek basin, is presently served by an extended aeration plant just north of Rock Springs Road. Pole Bridge Creek enters South River from the north about a mile upstream of Klondike Road. Conley Creek, a tributary entering from the south, receives at its headwaters effluent from the reatment facility at the Atlanta Army Depot. From available records for the interval October, 1968, to September, 1969, average daily flows at this plant were 0.305 mgd, while BOD's averaged 220 mg/l in the influent and 8.5 mg/l in the effluent.

Also within the upper basin, in Clayton County, is the Rock Cut Road System, presently utilizing an aerated lagoon for treatment of wastes. At least four more smaller treatment facilities are known to exist in DeKalb County, on the south side of the basin, upstream of Snapfinger Creek plant location.

Pollution Load From Wastewater Treatment Plants

The average daily pollution load upon the South River from the Intrenchment Creek and South River wastewater treatment plants is 14,540 pounds BOD. Bypassing procedures at each of the plants, although intermittent, add an average of 2,520 pounds per day or 17 percent of the treated effluent pollution load. However, bypassing does not occur on a daily basis, so the effect upon the South River is far more pronounced when it does occur, as discussed in Section X.

Bypassing of flows from both the Intrenchment Creek interceptor and the Sugar Creek trunk adds approximately 1,850 pounds BOD per day to the 8,800 discharged as treated effluent at the Intrenchment Creek wastewater treatment plant, amounting to an increase of 21 percent. Bypassed flows from both sections of the South River wastewater treament plant add 670 pounds BOD per day to the 5,740 discharged as treated effluent, yielding an increase of 12 percent.

As an intrastate stream, the South River does not fall under federal water quality standards. State standards have been established however. The content of these standards and the reduction in pollution load required to attain them is discussed in Section XI, CONDITIONS IN THE STREAM.

SECTION VIII

OTHER SOURCES OF POLLUTION

Overflows from Temporarily Inoperative Pumping Stations

Topographic conditions in the upper South River basin are such that sanitary flows from practically all areas within its boundaries can reach the treatment plants by gravity. However, two pumping stations exist and are operated within the basin. One of them, known as Constitution Road pumping station, receives sewage from a drainage area of about 408 acres within the basin of the Federal Prison branch. The Constitution Road pumping station discharges through an 8–inch force main into the 21–inch Federal Prison outfall that flows into South River plant.

The second pumping station is located at Jonesboro Road within the site of the South River treatment plant itself. Purpose of this station is to lift part of the sewage that arrives at the plant from the south and southwest at a lower elevation, to the higher elevation of the 42–inch South River sewer entering from the northwest. Sewage requiring pumping arrives through the Bromack Drive, Hapeville, and Jonesboro Road (south) outfalls.

When the Constitution Road pumping station becomes inoperative, bypassed raw sewage is discharged into Federal Prison branch at a point about 3,500 feet upstream of its confluence with South River.

No records were kept by the City on times when bypassing of this pumping station occurred during 1969. The pumping station at Jonesboro Road experienced mechanical trouble from January 13 to March 7, 1969, causing a partial bypassing of flows arriving at the plant. It operated uninterruptedly for the rest of the year.

Pollution From Industrial and Other Sources

A large number of industries are found within the upper South River basin, many of them along the ridges that establish the boundary, where most railroad tracks are located. Some of these industries contribute untreated wastes to the system, while others provide preliminary treatment of some kind before discharging them. It is the policy of the Atlanta Metropolitan Sewer System that all industrial wastes be accepted into the sewers and provided with further treatment after suitable pretreatment by industry.

A complete survey of the industrial waste situation within the basin is not feasible for this study, due to the number and variety of industries involved, and complexity and variability of some industrial processes. As a sample, a list for the McDaniel Street subbasin is included in Appendix D. Information was obtained by field survey. Illegal discharges of wastes that would have required pretreatment probably occur from time to time. It would be practically impossible to find and assess many of these sources of pollution.

Conductivity recordings as well as sampling and observation at some of the study stations have shown eventual discharges of pollutants of unknown origin reaching the streams. Identification of specific pollutants in the sewage and of possible sources has not been made since it was not justified for the purpose of the study.

In a field survey of the headwaters of the south branch of the river it was concluded that during heavy rains, overflow from certain holding ponds for pretreatment of industrial wastes could possibly reach the stream. To ascertain conditions like these, specific inspections would be required during storms.

Toxicity studies carried out on a number of samples taken at selected locations have confirmed presence of toxic waste in variable concentrations. Variability of conditions is such, however, that no quantitative statement can be made in connection with industrial waste discharges.

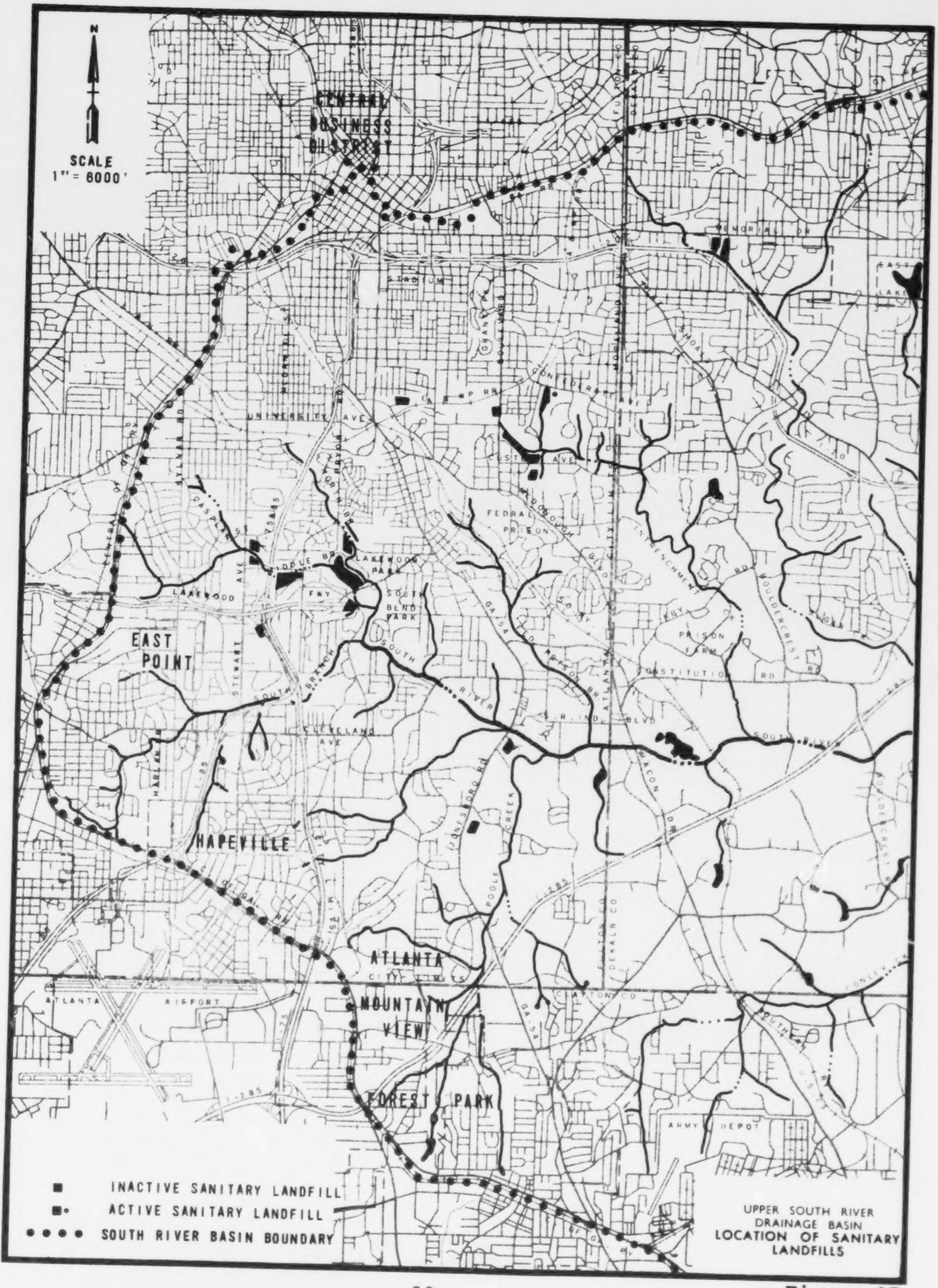
Pollution from Sanitary Landfills

Fifteen landfills are found within the upper South River basin, but only one is presently active. Location of these landfills appears in Figure 37. It can be noticed that most of them are adjacent to streams.

The active landfill, located adjacent to the Confederate Avenue combined sewer overflow, has been extended south on the west side of the sewer and along the open concrete canal recently built at the overflow point. Natural ground at this location is composed mainly of impervious clays and slopes gently in the direction of Intrenchment Creek. Both conditions facilitate pollution from material in the landfill reaching the stream with storm runoff.

An assessment of pollution from this source has not been successful; however, it has been observed that during heavy rains, surface runoff from the area carries clay and other finely suspended materials into the stream. It might be expected that leachates from landfills should also reach the stream under these conditions, contributing increased pollution loads to the overflows. Stronger liquors from deeper fills have been found by other investigators to contain amounts of BOD in the 2,000 to 4,000 mg/l range. However, volume and strength are highly dependent upon fill constituents, soil percolation, landfill construction, and many other factors. At dry weather conditions, no seepage or flows from the landfill have been observed to reach the canal or the creek. A significant amount of rainfall can be expected to be absorbed by landfill materials before any leaching occurs.

Decrease of the quality of overflows due to contributions from this source is not easily estimated. Grab sampling procedures are unsafe, due to the very high flow velocities in the canal adjacent to the landfill and due to the lack of a suitable sampling point along the banks of the canal. The latter constraint also limits use of an automatic sampling device. Furthermore, observation of storm flows at the land fill site indicated that dilution of small leachate volumes in these large flows would yield changes in water quality of so small a magnitude that a separate, detailed study would be required to detect them. Personnel,


N
SCALE
1" = 6000'
CENTRAL
BUSINESS
DISTRICT
STADIUM
UNIVERSITY AVE
MEMORIAL DR
EAST LAKE
CONFEDERATE AVE
CUSTER AVE
FEDRAL PRISON
LAKEWOOD PARK
SOUTH BEND PARK
LAKEWOOD FWY
EAST POINT
HAPEVILLE
CLEVELAND AVE
SOUTH RIVER
PRISON FARM
CONSTITUTION RD
BOULDERCREST RD
JONESBORO RD
POOLE CREEK
I-285
I-75
I-85
ATLANTA
CITY LIMITS
MOUNTAIN VIEW
FOREST PARK
ATLANTA AIRPORT
ARMY DEPOT
CLAYTON CO
DEKALB CO
FULTON CO
■ INACTIVE SANITARY LANDFILL
■• ACTIVE SANITARY LANDFILL
● ● ● ● SOUTH RIVER BASIN BOUNDARY

UPPER SOUTH RIVER
DRAINAGE BASIN
LOCATION OF SANITARY
LANDFILLS

Figure 37

equipment and financial contraints contributed to the necessity for omitting this separate study, thereby permitting effort to remain focused on the primary goal of this report – an assessment of combined sewer pollution.

Pesticides and Other Chemicals

Presence of chemicals, possibly pesticides, has been suspected at times by odor of water at certain points in the stream. However, adequate determinations have not been performed, since the laboratory was not set up for this type of analysis. It was not physically or financially feasible to have pesticide determinations performed elsewhere within a reasonable distance.

Contributions from Vacant Lots, Agricultural Lands, Air Pollution

The percentage of vacant lands is practically zero in the north sections of the basin where combined sewers exist, but becomes significant to the south, where development is still taking place. Contributions from these sources must be assessed as a whole from conditions at various stations on the stream for different flow regimes.

Downstream of Intrenchment Creek wastewater treatment plant, and adjacent to the stream, is located the Atlanta Prison Farm.

Pollution from cattle, feed lots, agricultural uses, and other sources on this farm might add to that reaching the stream, but conditions in this section are normally so critical that isolation of individual pollution sources is extremely difficult.

It is known that rain scavenges air pollution. In a highly industrialized area, as is the case in all of the upper South River basin, significant contributions can be expected to originate at this source. Air pollution will add pollutants to storm water runoff and combined sewer overflows. Assessment of this source in presence of many others is also an almost impossible task.

Due to the difficulty of isolating pollution from such sources to determine the significance of each, this study has included all such effects in the determination of equations for quality of storm runoff and combined sewer overflows.

SECTION IX

EFFECT OF ZONING AND LAND USES ON POLLUTION LOADS

Zoning Regulations

Zoning regulations of the City of Atlanta establish twenty different zoning districts. District uses and regulations appear on maps covering the corporate area to the scale of 1 inch equals 1,200 feet. These maps were utilized in the preliminary assessment of differences in land use between drainage areas or subbasins that could be monitored for detailed study.

For simplicity in selection and correlation with pollution data, the City's zoning districts were grouped into four general types for preliminary work. These types included low and high density residential, industrial, and commercial districts. Each drainage subbasin selected for pollution load monitoring was subdivided into the four general zoning types, and areas within each type were calculated.

Land Uses

Zoning districts include built up and undeveloped areas indiscriminately. Thence, actual land uses, and not zoning districts, had to be correlated with pollution data for a realistic approach. A detailed determination of land uses within each selected subbasin was undertaken when drainage maps became available. In this work aerial photomaps were utilized and land uses were established block by block, obtaining additional information in the field as required. These photomaps have the same scale (1 inch equals 200 feet) as the study's drainage maps, and it was possible to overlay detailed land uses on the same maps where drainage boundaries were carefully established.

Areas under each of six general land use classifications were determined on these maps. It is not practical to include these maps in the report, since large scale reductions would be required and a large number of sheets is involved.

A sample of the typical land use determination performed, taken from Sheet P–16 within McDaniel drainage area, can be seen in Appendix D.

Within the study area is located the Atlanta Model Neighborhood which is being developed under the Demonstration Cities and Metropolitan Development Act of 1966. The Model Neighborhood area occupies 3,130 acres, nearly all of which is served by the combined sewer system. For the purpose of the investigation, however, present land uses were obtained for correlation with pollution data, independent of future plans for parts of the study area.

Figures 38 through 41 are simplified maps of each of the smaller drainage areas studied in detail, in an attempt to correlate pollution loads to land uses. Drainage boundaries and land uses have been shown on each area to permit easy visual comparison.

For land use determinations in other drainage areas outside the city limits of Atlanta, recent aerial photographs and information from Atlanta Region Metropolitan Planning Commission (ARMPC) were utilized.

In general, all of the area within the upper South River basin is highly industrialized. Although differences in land uses do exist between selected subbasins, they are not highly significant, and percentages of land for industrial uses are similar in all. These conditions hinder possible conclusions concerning effect of land uses on quantity and quality of runoff, as found in the analysis of pollution data.

Calculated percentages of land uses within each drainage subbasin studied in detail are grouped in Table 21 below. Figures for subbasins 1, 2, 3, and 5, entirely within the area mapped for the study, are based on the detailed determination described before. Calculations for areas 4 and 6 had to be based on less accurate maps and uncontrolled aerial photographs. Low density land use was considered to apply to all single family dwellings and duplexes, whereas high density land use included all apartments. For both uses, the number of persons per dwelling varied widely.

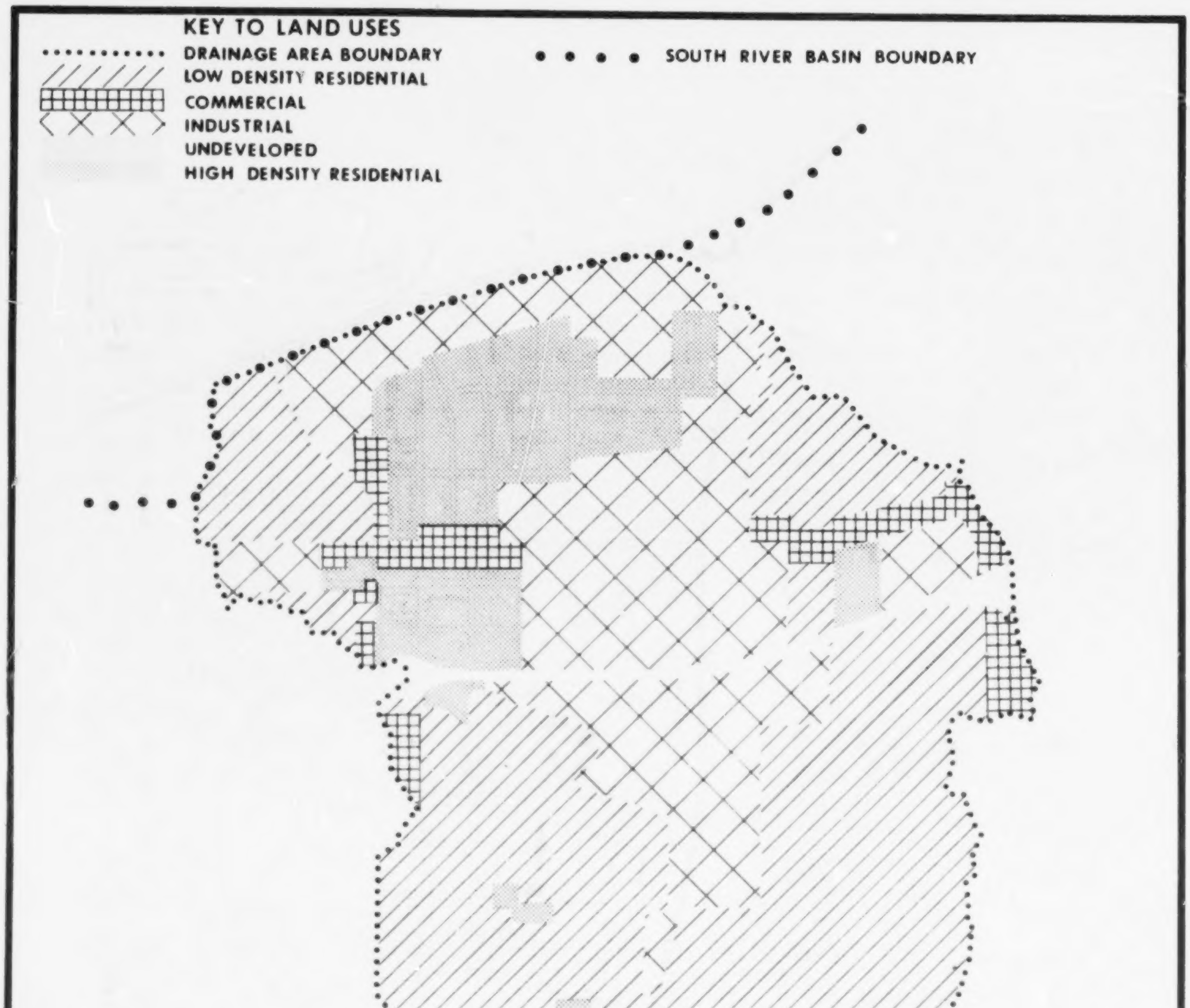
TABLE 21

UPPER SOUTH RIVER DRAINAGE BASIN
LAND USES WITHIN SUBBASINS

(Percent)

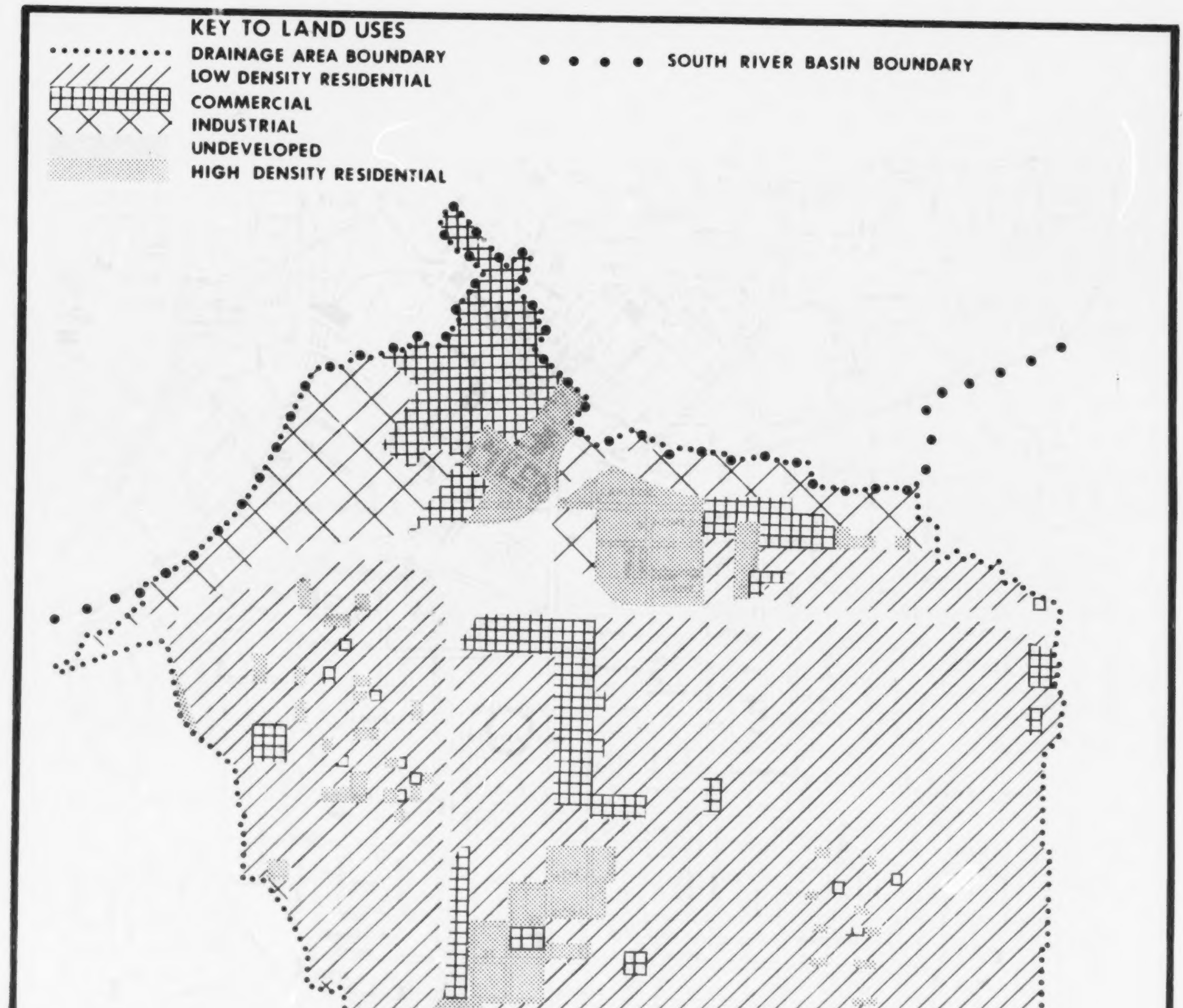
Land Use	Confederate 01*	Boulevard 02*	McDaniel 03*	Harlan 04	Casplan 05*	Federal Prison 06
	Subbasin Designation and Number					
Commercial	4.50	8.11	1.80	8.00	2.49	5.12
Low density residential	54.72	51.77	56.50	47.70	60.59	34.92
High density residential	13.03	12.37	4.58	9.20	--	0.58
Industrial	25.35	19.27	35.64	24.70	28.45	20.41
Undeveloped – open	--	2.13	1.48	10.40	8.47	36.52
Interstate ROW	2.40	6.35	--	--	--	--
Total	**100.00**	**100.00**	**100.00**	**100.00**	**100.00**	**100.00**

*Figures based on study's drainage mapping, aerial photomaps, and field verification.


KEY TO LAND USES
DRAINAGE AREA BOUNDARY
LOW DENSITY RESIDENTIAL
COMMERCIAL
INDUSTRIAL
UNDEVELOPED
HIGH DENSITY RESIDENTIAL
SOUTH RIVER BASIN BOUNDARY
Overflow
GRAPHIC SCALE
1500
0
1500 ft

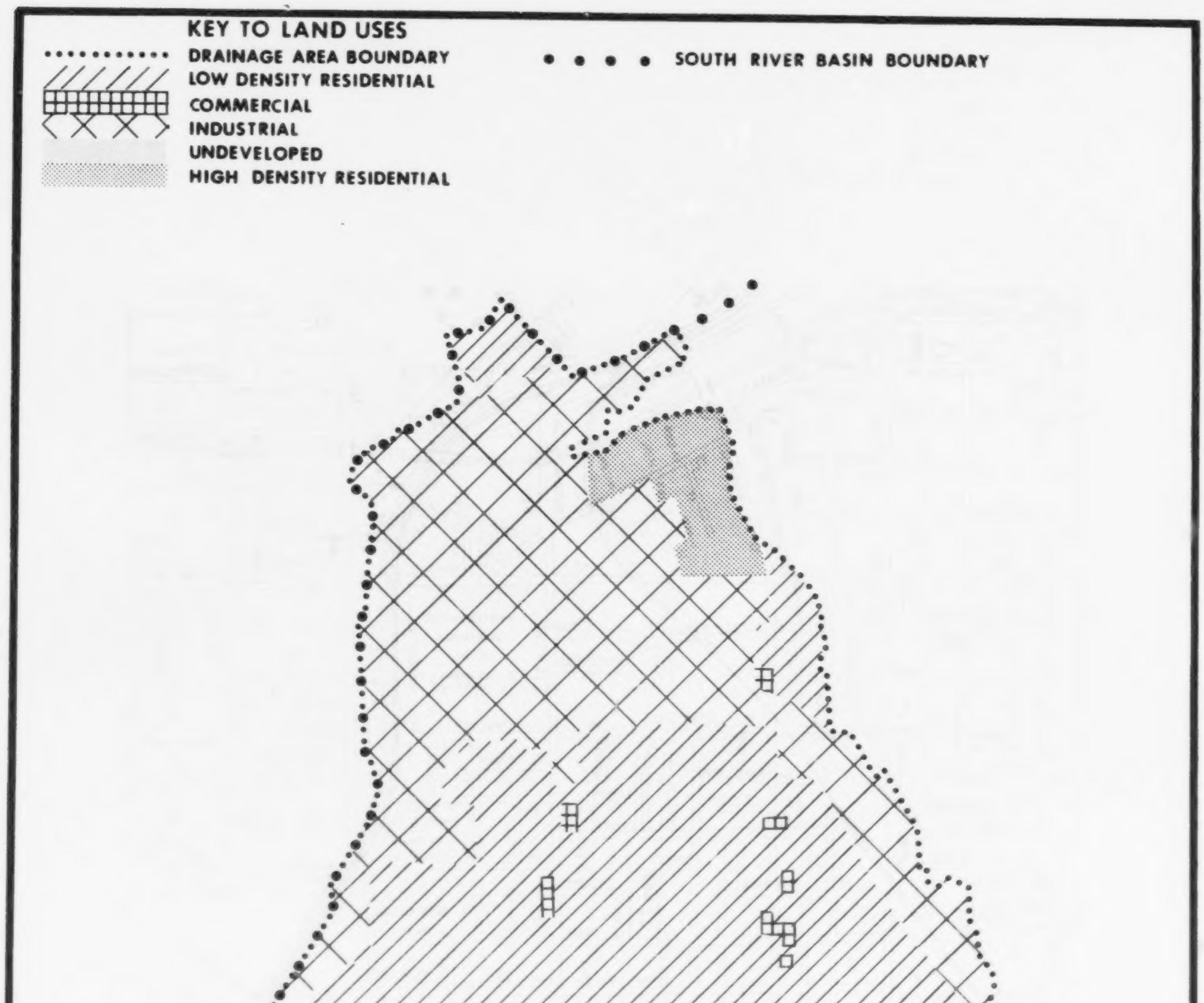
DRAINAGE AREA NO.1
CONFEDERATE AVENUE
COMBINED SEWER OVERFLOW

Figure 38


KEY TO LAND USES
DRAINAGE AREA BOUNDARY
LOW DENSITY RESIDENTIAL
COMMERCIAL
INDUSTRIAL
UNDEVELOPED
HIGH DENSITY RESIDENTIAL
SOUTH RIVER BASIN BOUNDARY
Overflow
GRAPHIC SCALE
2000
0
2000 ft.
N

DRAINAGE AREA NO. 2
BOULEVARD
COMBINED SEWER OVERFLOW

Figure 39


KEY TO LAND USES
DRAINAGE AREA BOUNDARY
LOW DENSITY RESIDENTIAL
COMMERCIAL
INDUSTRIAL
UNDEVELOPED
HIGH DENSITY RESIDENTIAL
SOUTH RIVER BASIN BOUNDARY
Overflow
Gaging Station
GRAPHIC SCALE
1500
0
1500 ft.
N

DRAINAGE AREA NO.3
McDANIEL ST.
COMBINED SEWER OVERFLOW

Figure 40

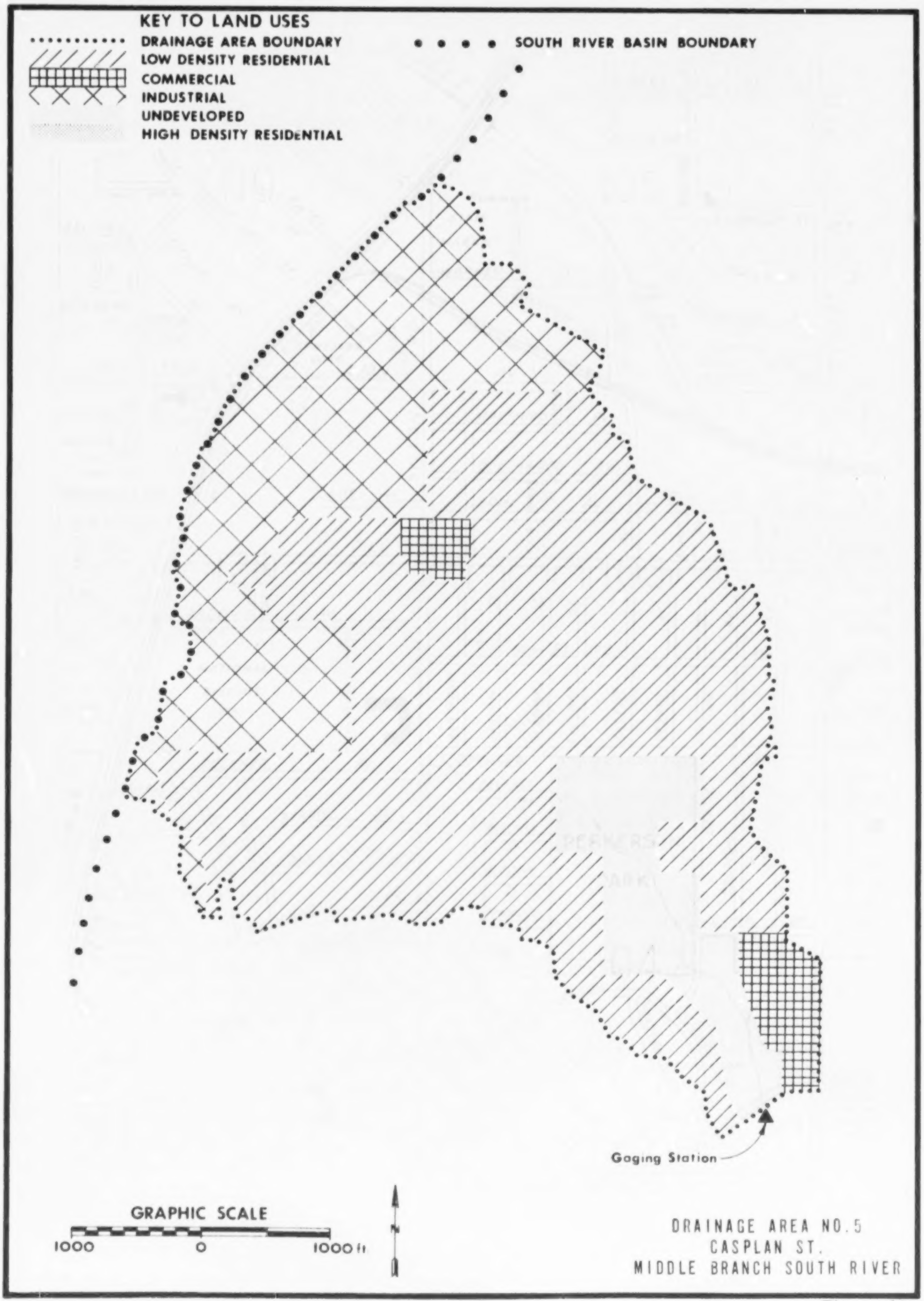

KEY TO LAND USES
DRAINAGE AREA BOUNDARY
LOW DENSITY RESIDENTIAL
COMMERCIAL
INDUSTRIAL
UNDEVELOPED
HIGH DENSITY RESIDENTIAL
SOUTH RIVER BASIN BOUNDARY
Gaging Station
GRAPHIC SCALE
1000
0
1000 ft
N
DRAINAGE AREA NO.5
CASPLAN ST.
MIDDLE BRANCH SOUTH RIVER

Figure 41

Effect of Land Uses Upon Quality and Quantity of Dry Weather Flows

Dry weather flow characteristics at each of the six stations were monitored during the period of study. The results of the BOD analyses are presented in Table 22.

TABLE 22

Station		BOD – Dry Weather Flow	
		Mean	Standard Deviation
11	Confederate Avenue	462	169
21	Boulevard	194	78
31	McDaniel Street	161	71
4	Harlan Drive	14	4
5	Casplan Street	17	12
6	Federal Prison Branch	18	12

The high BOD's associated with the Confederate Avenue interceptor reflect the extent of industrial activity in that area. Although industrial land use, at 25 percent of the subbasin area, was not substantially different than in the other two combined sewer areas, the waste BOD's are significantly higher due to the types of industries concentrated in the subbasin. Overflows at Confederate Avenue have been observed to change color due to introduction of textile dyes to the waste collection system. Such dyes often have high BOD's and yield deleterious effects upon receiving waters.

Dry weather flows at stations 11, 21, and 31, the Confederate Avenue, Boulevard and McDaniel Street interceptors, averaged 5, 12, and 6 cfs, respectively. No correlation was evident between land use and dry weather flow quantity, either when expressed as gallons per capita per day or as cfs per acre.

As described previously, land uses for all six areas were not substantially different, thereby hindering correlation of dry weather flow characteristics and land use. Analysis of the data from a limited number of subbasins suggests that such a correlation may be unrealistic for combined sewer areas receiving industrial waste. For storm sewer areas it may be feasible; however, data obtained from this study was insufficient to establish any definite relationship due to the similarity of land uses for each of the three storm sewer areas. This point will be discussed further in the next paragraph.

Effect of Land Uses Upon Quality and Quantity of Storm Runoff

Storm runoff at all six subbasins was monitored during the period of study; however, as discussed in Section IV, areas 2, 3, and 5 were selected for detailed analysis. Since dilution of influent industrial and domestic waste constitutes the primary source of BOD in combined sewer overflows, it is reasonable to expect that quality of storm runoff should reflect dry weather flow conditions. This was found to be true for all three combined sewer flows, although the high BOD's associated with overflows at station 1 were of limited statistical significance due to the lack of data at high flow rates.

Runoff coefficients for areas 2 and 3 were identical at 0.42 while for area 5 the coefficient was 0.56. All three were higher than the 0.39 average of all six subbasins. The similarity of land uses and the limited number of subbasins precluded correlation of land uses with runoff quantities from storm events.

A statistical analysis was performed to determine those variables contributing significantly to a relationship between land use and pollution load. Initially, variables under consideration consisted of the six land use classifications included in Table 10–1. It was later determined that replacement of high and low density residential classifications by a single population density variable improved the correlation. A linear relationship was assumed since the limited data did not warrant more refined analysis. Pollution loads expressed as pounds BOD per acre per hour of rainfall were obtained from Tables 7 and 16. The storm sewer area contributing to station 5 was assumed to be a combined sewer area with zero tributary population.

For all but three of the 14 synthetic storms, population density was the significant variable, while all others were rejected. For the remaining three, undeveloped land was selected. The relationship would be substantially improved by inclusion of pollution load data from additional subbasins, especially from those with widely varying land uses. Table 23 presents the results of the analysis from which an equation may be derived for any chosen storm, of the form:

$$Y = a + bX_1 + cX_2$$

where Y = pollution load (pounds BOD/acre/hour of rainfall)

X_1 = population density (persons/acre)

X_2 = undeveloped land (%)

TABLE 23

Storm Frequency	Rainfall Duration (hours)	a	b	c
2 weeks	1	3.250		–0.247
2 weeks	2	0.785	0.057	
2 weeks	5	0.330	0.025	
1 month	1	3.796		–0.190
1 month	2	1.278	0.073	
1 month	5	0.629	0.030	
1 month	10	0.315	0.020	
6 month	2	3.629	0.052	
6 month	5	1.260	0.034	
6 month	10	0.720	0.026	
1 year	2	3.905	0.057	
1 year	5	2.923		–0.162
1 year	10	0.880	0.027	
1 year	20	0.456	0.018	
		Average	**0.038**	**–0.200**

This table presents a base upon which considerable subsequent work may be developed, as discussed in the next paragraph. Certain hypotheses may be developed from these results and previous observations. For combined sewer areas, land use may not be the appropriate primary variable upon which to base pollution load estimates. For all but initial scouring flows and high flow rates, pollution characteristics depend primarily upon dilution of domestic and industrial waste entering the system. The strength of this waste determines the BOD concentration in the overflow. The pertinent parameter for correlation would therefore be the population equivalent of the domestic and industrial waste, which would entail a detailed analysis of industrial waste discharges within each combined area. Storm sewer areas and combined sewer areas at high flow rates yield BOD concentrations that may well depend upon land use more than upon population; however, further data will be required to determine the nature of the relationship.

Recommendations for Further Study of Land Use Effects

With very little additional field work, high frequency response characteristics of three additional subbasins in the study area may be determined, thereby permitting inclusion of already available data in the correlation of land use and pollution load. Doubling the sample size will increase significantly the statistical validity of conclusions concerning such a correlation. It would also permit determination of the form of equation most accurately describing the relationship between pollution load, storm frequency and rainfall duration.

With this equation, it may be possible to standardize the results of studies across the United States, to remove all rainfall dependent properties prior to correlation with land use effects.

SECTION X

COMPARATIVE ASSESSMENT OF POLLUTION SOURCES

Assessment of Pollution Sources During Dry Weather Flow

Effluent from the Intrenchment Creek and South River wastewater treatment plants constitutes the major source of pollution during dry weather flow. With average daily discharges of 8,800 and 5,740 pounds of BOD, respectively, these sources contribute a significant fraction of the total annual pollution load upon South River from the study area. the impact upon the stream may be judged from the resultant average dissolved oxygen profile for 1968–69, shown in Figure 42. This information was compiled by the Georgia State Water Quality Control Board (1970) and was based upon data obtained by that organization and the City of Atlanta. The average critical point is reached near Panthersville Road, about two miles below Bouldercrest, with a dissolved oxygen deficit of 4.35 mg/l. The proximity of the critical point to the headwaters area reflects a number of pertinent factors, most significant among which is the relatively large fraction of dry weather stream flow attributable to sewage treatment plant effluent. Approximately 75 percent of the flow at Bouldercrest originates at the wastewater treatment plants during dry weather periods. The resulting low flow rates yield low velocity and high reaeration rates. Furthermore, the deoxygenation rates of 0.20 for trickling filter effluents and 0.08 for activated sludge effluents assumed for this study, tend to yield rapid utilization of dissolved oxygen.

The minimum dissolved oxygen level in South River during 1968–1969 averages 1.9 mg/l as compared with the 3.9 mg/l average level. The minimum also occurred near Panthersville Road.

Other pollution sources are of minor importance. BOD's associated with the small dry weather flows from storm sewered areas have little impact upon the stream when compared with effluent from the treatment plants. Intermittent bypassing of raw sewage from the pumping station at Constitution Road, which has been observed during the study, may significantly affect water quality; however, no data are available to judge the extent of impact.

Assessment of Pollution From Storm and Combined Sewer Areas

A comparison of BOD loads from storm and combined sewer areas is shown in Figure 27, Section V. These curves indicate that annual average BOD loads from storm sewer areas are approximately 55 percent of those from combined sewer areas.

It must be noted that these pollution loads are based upon average flows in the combined sewers. A storm occurring during the peak annual flow that might be expected on an

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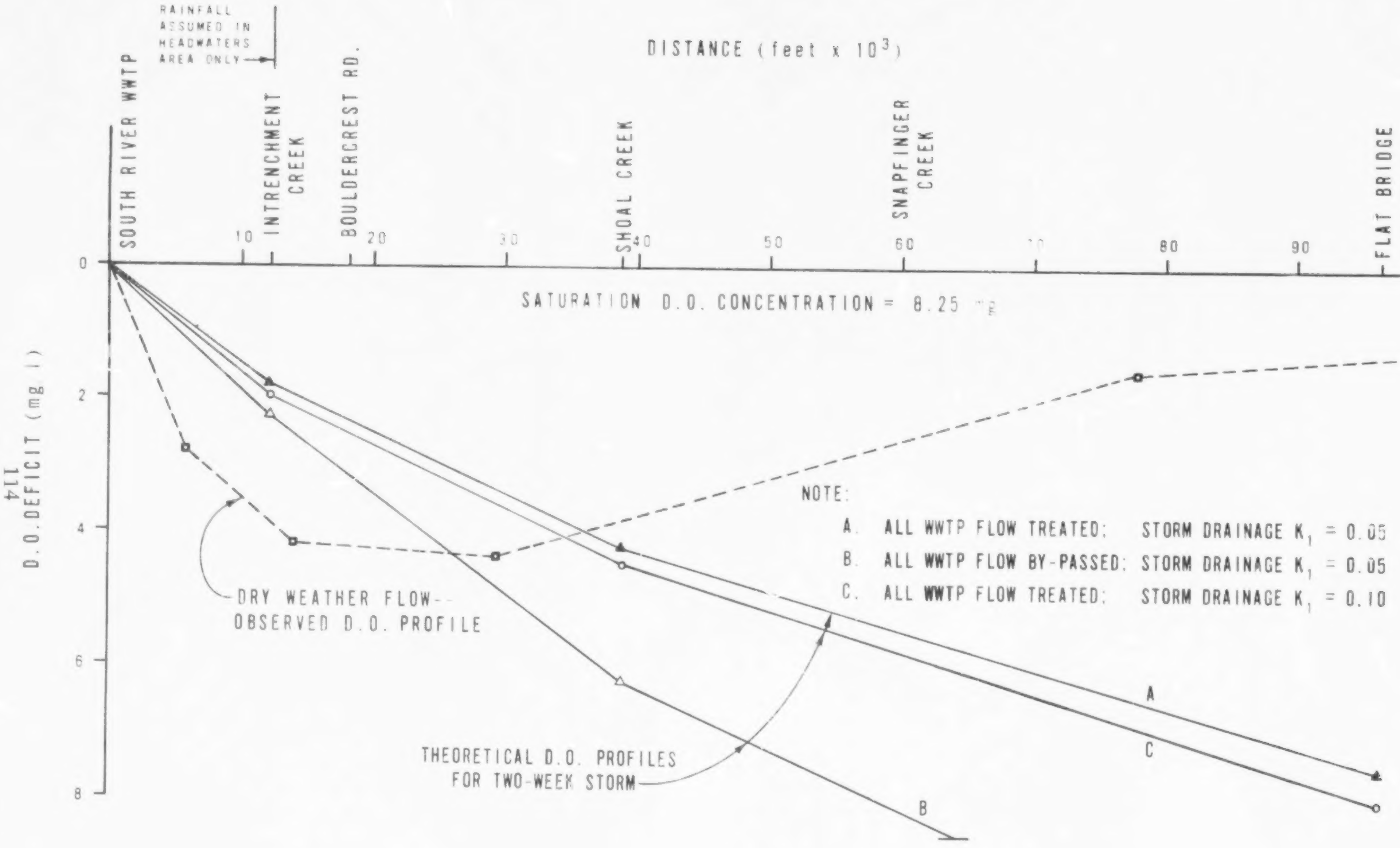

THEORETICAL ANNUAL AVERAGE DISSOLVED OXYGEN PROFILES IN SOUTH RIVER FOR TWO-WEEK STORM

Figure 42

afternoon in May, would yield BOD concentrations and pollution loads 56 percent higher than the average, whereas a similar storm occurring before dawn on a Sunday in February would yield only 56 percent of the average pollution load. Therefore ratios between pollution from storm and combined sewer areas actually depend upon the exact time of year.

Assessment of the Impact Upon the South River of Pollution Loads from Storm Runoff

Calculation of the theoretical dissolved oxygen profiles of Figure 42 revealed the relative importance of the various sources of pollution during runoff events. The profiles should be interpreted only with full understanding of the assumptions used in their derivation. In order to assess the impact of runoff from the study area upon the South River, it was assumed that the storm occurred uniformly over the headwaters only, down to the confluence of Intrenchment Creek and South River. Lateral inflow to the river below this point was assumed equal to that expected under dry weather conditions. The storm applied to the study area was a two–week storm of two–hour duration, with a peak at one–quarter of the duration. Treatment of intercepted flow was assumed for two of the profiles, while in the third it was assumed that all treatment plant flow was bypassed.

Deoxygenation rates were obtained from a study by Schroepfer et al., (1960) of the Mississippi River near St. Paul, Minnesota, as listed in Table 24.

TABLE 24

Waste Description	Range of $K_1(20^{\circ})$	Average
Untreated Sewage	0.40 – 2.61	1.0 (4,8,12 hours)
Activated Sludge Effluent	0.03 – 0.13	0.08
Trickling Filter Effluent		
High Rate	0.18 – 0.22	0.20
Standard Rate	0.18 – 0.24	0.21
Unpolluted River Flow	0.00 – 0.15	0.06

Generally accepted values of K_1 for untreated sewage of approximately 0.25 represent rates based upon 1, 3, and 5–day BOD's, whereas Schroepfer et al., showed that initial rates for time periods less than 12 hours are much higher. For storm runoff from urban areas, deoxygenation rates were assumed to range from 0.05 to 0.10 although this estimate requires verification. The sensitivity of the dissolved oxygen profile to variations within this range is included in Figure 42. All temperatures were assumed to be 24° C, the average occurring during 1968–1969. Velocities of three fps were assumed along the whole river, in order to maintain the observed lag time of 9–12 hours between commencement of rainfall and arrival of the flood wave at Klondike.

The river was divided into five reaches, in each of which steady, uniform flow was assumed to occur, at flow and pollution rates equal to the average occurring during the duration of runoff within the tributary area. A far more sophisticated flood routing computer program, requiring detailed information from the entire South River basin, will be desirable for future studies in order to assess the exact impact of pollution sources and abatement alternatives upon the South River under dynamic conditions.

The approximation to the actual hydraulic response of the river, as utilized in this study, is considered to be sufficient for assessing the relative impact of various sources of pollution upon the water quality of the South River.

Average reaeration rates in the South River were calculated for each of the five reaches, utilizing hydraulic properties and equations determined by Wallace and Reheis (1969). Data was augmented by information obtained in this study. Although the equations were derived for dry weather flow conditions, they are considered to represent storm flow conditions more accurately than any alternative procedure, other than measurement under such conditions.

Combined sewer overflows at Confederate Avenue and Boulevard increase the ultimate BOD of the upper end of Intrenchment Creek by 14 to 52 percent above the level to be expected from storm runoff alone. The range reflects the sensitivity to variation of K_1 for storm runoff from 0.05 to 0.10 respectively. However the maintenance of any stream standard of water quality in the South River will require consideration of biochemical oxygen demand during periods of time less than one day. Due to the high initial deoxygenation rates associated with untreated sewage, the ultimate BOD is less significant a gage of pollution impact than a short term BOD that reflects these high rates. The only such measure commonly available is the 5–day, 20° C BOD, which is increased at the upper end of Intrenchment Creek by 130 percent above the level to be expected from storm runoff alone. The analysis suggests, therefore, that the impact of combined sewer overflows upon the receiving stream may be due to both the increase in volume of biodegradable matter discharged and the increase in the average deoxygenation rate above the level for storm runoff alone.

Flow entering Intrenchment Creek from the Confederate Avenue and Boulevard overflows is diluted by lateral inflow until reaching the Intrenchment Creek wastewater treatment plant. Overflows at the McDaniel Street interceptor increase the 5–day, 20° C BOD by approximately 130 percent, although the impact is rapidly dissipated by dilution from lateral inflow. By the time flow reaches the South River wastewater treatment plant, the increase in 5–day, 20° C BOD due to combined sewer overflow is only 7 percent.

Another effect of combined sewer overflows upon a receiving stream has been reported by Engineering Science, Inc. (1967) in a study of combined sewer pollution in San Francisco. They observed that combined sewer overflows have a considerable impact on the bacterial quality of receiving waters. However, coliform MPN's were estimated to decrease by 90 percent within approximately 12 hours, slightly greater than the travel time to Klondike.

The relative impact of combined sewer overflows and storm runoff for other than two–week, two–hour storms may be estimated from Figure 26. The impact for lower frequency storms decreases; however, such storms are of lesser significance in combined sewer studies.

Wastewater treatment plant effluents during runoff events, including treatment of intercepted flows, have little impact upon the South River, in contrast to their significant influence during dry weather flows. Calculations indicated that treated effluent at the Intrenchment Creek wastewater treatment plant may decrease the average ultimate BOD at that point.

Bypassing of all flows, including intercepted combined sewer flows, has a severe impact upon the South River. As described in Section VII, this procedure is common during periods of storm runoff. It is also more severe in its pollution impact than discharge of the same combined sewer flow at the interception points, because additional sewage enters the interceptor lines before bypassing at the treatment plants. Bypassing therefore adds considerable volumes of untreated waste to the flows from combined sewer areas.

The high rate of deoxygenation for untreated waste during the first 12 hours causes an increase of approximately 33 percent in the average deoxygenation rate along the entire South River to Klondike. Although bypassed flow rates at the South River wastewater treatment plant tend to be lower, the impact upon the stream is greater due to the high percentage of sanitary waste in these flows. Under these limited conditions, the increase in deoxygenation rate would cause the South River to become anaerobic at a point approximately 8 miles upstream of Klondike.

It is emphasized that these conclusions are theoretical and apply only under the conditions outlined at the beginning of this section. It is unlikely that all wastewater treatment plant flows would be bypassed for a storm of this size. A normal storm of this frequency would cover not only the headwaters, but probably all, or a large part of, the South River basin, thereby causing considerable dilution of BOD concentrations. The added inflow would also increase stream velocity, tending to move the dissolved oxygen profile downstream. The profiles of Figure 42 may therefore be considered conservative in their representation of normal stream conditions during a storm.

The only difference in the calculations of curves A and C of this figure is the variation in the assumed value of K_1 for storm drainage. Both of these curves assume treatment of all waste and intercepted combined flows. Application of the higher value of K_1 to the case where all treatment plant flows are bypassed, would cause South River to become anaerobic further upstream of the point shown on curve B.

Valid assessment of the relative impact of individual pollution sources upon the South River may be obtained from the calculations despite uncertainty as to the exact value of some variables. However, accurate prediction of dissolved oxygen levels in the South River subsequent to treatment modifications will require a more detailed analysis of the physical and hydrologic characteristics of the South River basin than was required for this study.

Present sources of storm pollution, in order of decreasing annual pollution load, are as follows:

		lbs BOD/year
1.	Storm drainage from urban areas tributary to Bouldercrest Road (22,042 acres).	5,577,000
2.	Combined sewer overflows at Confederate Avenue and Boulevard (3,550 acres).	1,633,000
3.	Bypassing of flows from the Intrenchment Creek interceptor.	506,000
4.	Combined sewer overflow at McDaniel Street (968 acres).	445,000
5.	Intrenchment Creek wastewater treatment plant effluent.	185,000
6.	Bypassing of flows from South River wastewater treatment plant.	183,000
7.	South River wastewater treatment plant effluent.	146,000

Although important, annual pollution load is less significant than impact of individual storms upon the South River. Consideration of bypassing frequency, impact of overflows and bypassed flows upon average deoxygenation rates, bacterial effects and pollution loading rates, suggests that priorities among the seven pollution sources be shifted. In order of decreasing impact, the following arrangement appears reasonable:

		lbs BOD/year
1.	Bypassing of flows from the Intrenchment Creek interceptor.	506,000
2.	Combined sewer overflows at Confederate Avenue and Boulevard (3,550 acres).	1,633,000
3.	Bypassing of flows from South River wastewater treatment plant.	183,000
4.	Combined sewer overflow at McDaniel Street (968 acres).	445,000
5.	Storm drainage from urban areas tributary to Bouldercrest Road (22,042 acres).	5,577,000
6.	Intrenchment Creek wastewater treatment plant effluent.	185,000
7.	South River Wastewater treatment plant effluent.	146,000

SECTION XI

CONDITIONS IN THE STREAM

General

A discussion of conditions in the South River, its branches and tributaries at or near the headwaters, involves preliminary establishment of a point along the stream below which there is a possibility that actual conditions could be compared to some quality standard.

As discussed early in the report, the basin's headwaters lie within sewer systems, and extension of these systems in a downstream direction can be expected to occur with growth of the area. From a practical point of view, a few upstream sections of the stream system have to be considered as open storm sewers requiring encasement rather than polluted streams needing protection or treatment.

For these reasons a study of conditions along the stream must start at the Jonesboro Road bridge, a short distance above the outfall of the South River wastewater treatment plant. At this point, the stream has a definite base line flow and shows fairly good quality conditions during dry weather.

Dry Weather Flow Conditions

Dry weather flow conditions along the river, below the Jonesboro Road bridge, have been monitored for the past two years by the State Water Quality Control Board (SWQCB), as discussed before. These studies covered a total of seven stations distributed over forty–four miles of river length. An oxygen sag curve for part of this length is included in Figure 42.

For the present study, monitoring stations were established at the Jonesboro Road, Bouldercrest Road and Klondike Road bridges. These stations are shown in Figures 43 and 44. Although the length of time over which conditions in the river were monitored is shorter than the SWQCB period of record, results obtained at dry weather flow conditions were in fairly good agreement with values in their studies. Dissolved oxygen concentrations at dry weather flow conditions for the sewer stations monitored by SWQCB are included in Figure 42. Values are arithmetic averages of available results for each station.

Stream Survey Observations

Field inspection of conditions at the river's headwaters, as well as at some of the small tributary streams, revealed that during dry weather streams showing some base flow were rather clean. No flow was observed at first sections of streams originating at combined sewer overflows, such as the north branch near I–75 or Intrenchment Creek near Boulevard.

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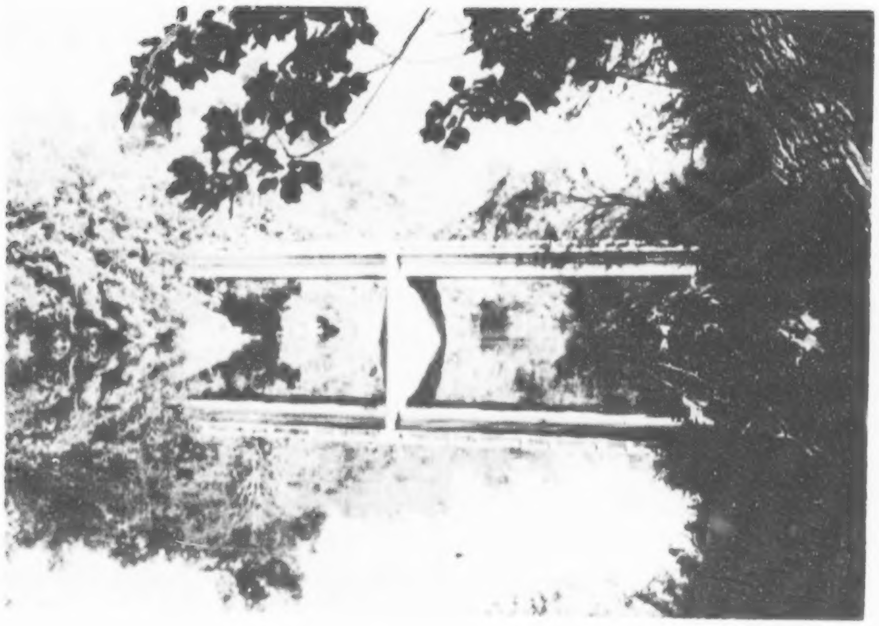
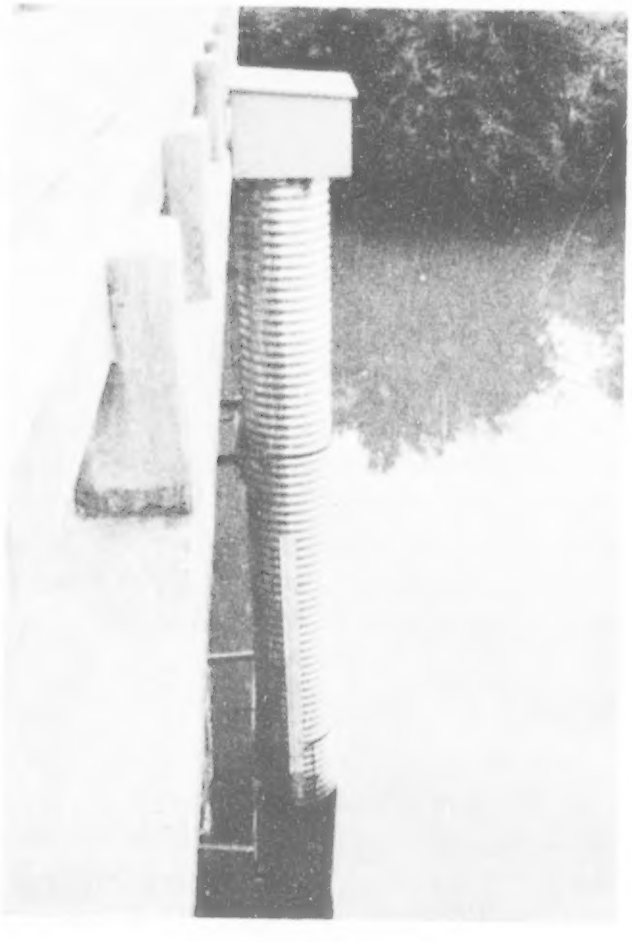


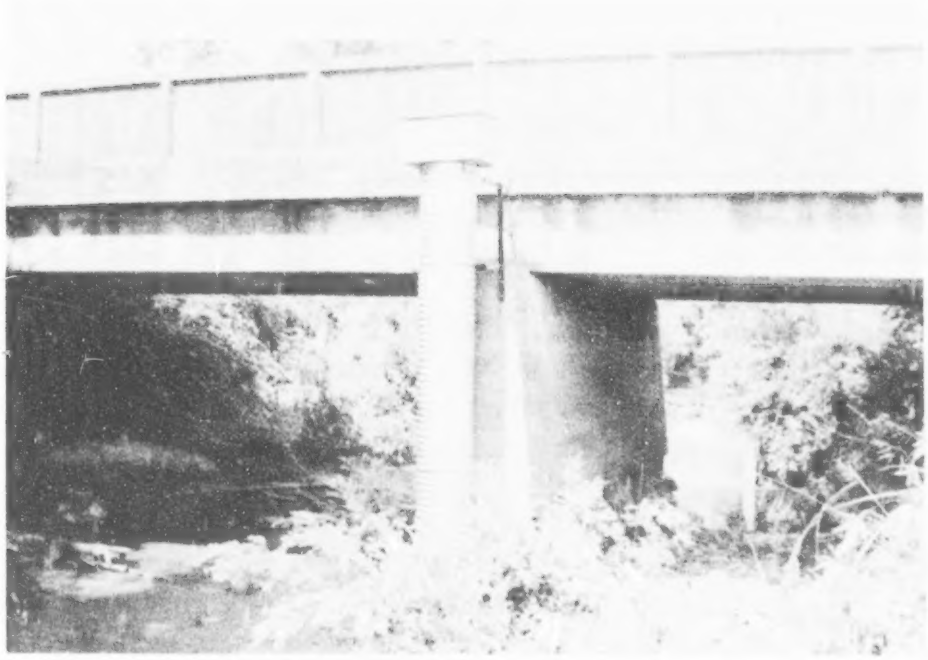
Gaging and Sampling Station



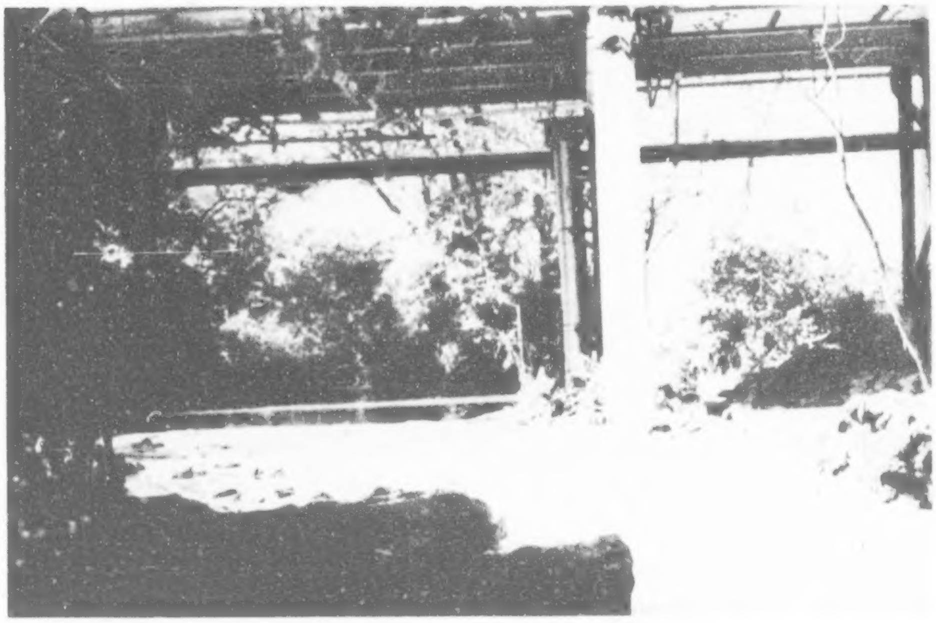
Flow Level Recorder

SOUTH RIVER
AT
KLONDIKE ROAD

Figure 43



Flow Level Recording Station
South River at Jonesboro Road



Flow Level Recording Station
South River at Bouldercrest Road

Figure 44

At Lakewood Avenue, the north branch had some base flow, was rather clean and had no odor. Measured conductivities were in the range of those found at water sources for the area, approximately 200 μmhos/cm.

The middle branch, near Casplan Street showed temperatures of 85° F during hot summer days. The stream generally was moving slowly, had no odor, low conductivity and showed some growth on the bottom.

The south branch was surveyed several times. Conductivities were usually high to very high at this stream. Some possible industrial sources were also inspected. Lagoons at Tennessee Corporation had conductivities exceeding 1,000 μmhos/cm. Values as high as 2,000 - 3,000 were observed where wastes from Allied Chemicals entered the stream.

Intrenchment Creek was observed to have some flow, no odor and clean appearance, with low condictivities, at Custer Avenue. Downstream of the wastewater treatment plant, at Constitution Road, its waters were dark reddish in color, had foul odor and rather high conductivities, with very low amounts of dissolved oxygen. Soapsuds were also observed frequently.

Poole Creek, entering South River from the south upstream of Forest Park Road, generally had a clean, free running appearance at dry weather conditions.

The main stream of South River was generally found to be clean at the Jonesboro Road bridge, upstream of the South River wastewater treatment plant outfall. Dissolved oxygen was found to be always above 6 mg/l at dry weather conditions, and no odors were noticed.

At Bouldercrest, continuous discharges of effluent from both treatment plants accounted for low dissolved oxygen values at dry weather flow conditions. Recorded conductivities were always high. Water was observed to maintain high color and turbidity. Flow consisted mainly of plant effluent, with BOD's usually in the 20–30 mg/l range, as a result of effluent dilution with base flow in the stream.

Storm Flow Conditions

Faster flow velocities and slower average deoxygenation rates for storm water were expected to move the critical point of the dissolved oxygen sag curve to a point further downstream from the dry weather critical point near Panthersville Road. Heavy pollution loads from urban storm runoff and bypassing procedures are diluted by lateral inflow along the South River, thereby offsetting the impact upon water quality.

Observations at the Klondike Road monitoring station indicated that flow at this point becomes anaerobic under certain conditions. Difficulty was encountered in continuous monitoring of DO and conductivity since the low velocities of dry weather flows caused the probes to clog. However, the existing data indicate that a two–week storm uniformly distributed over the South River basin will cause anaerobic conditions in the stream at

Klondike Road for a brief period of time. A larger storm of one–year frequency may be expected to lower DO levels to approximately one mg/l, reflecting the higher velocity and dilution in the stream. Extended duration, low intensity rainfall causes DO at Klondike to vary around three mg/l.

Valid conclusions based upon the limited data are difficult. However, it appears that the large amount of pollution from very low frequency storms is rapidly flushed from the South River basin, exerting its major impact upon water quality below Klondike Road, where dilution from lateral inflow becomes significant. Higher frequency storms scour the same initial organic load from the combined sewers, but receive less dilution along the river. Velocities are lower and average deoxygenation rates are higher, causing greater impact upon the stream within the South River basin. Finally, very minor rainfalls, which constitute the majority of rainfall events in the Atlanta area, cause sufficient overflow to scour the combined sewers while adding little dilution along the river. Impact from these rainfall events is the greatest and most frequent. It is also the easiest to prevent, due to the relatively low volumes of overflow. Further verification of the observations would entail monitoring of water quality down to Jackson Lake.

Stream Uses and Water Quality Standards

Conclusions concerning alternative pollution abatement schemes should take into consideration not only the relative importance of various sources of pollution, but also the present and future uses of the stream and costs required to maintain the water quality standards associated with those uses.

As an intrastate stream, the South River falls under jurisdiction of the State Water Quality Control Board, which requires secondary treatment of all municipal wastes. At present the upper stream reaches are used solely for the transport of wastewater treatment plant effluent, which constitutes a significant fraction of the dry weather stream flow. In previous years expanding development of the study area has caused conversion of sections of the headwater channels to closed culverts. The trend is expected to continue in the future.

Future increases in pollution loading, during both wet and dry weather, will further deplete oxygen resources of the South River. Adequate planning on a regional scale should be implemented to ensure optimal future use of the stream by the Atlanta area and downstream communities. Such planning would pinpoint future competition for the stream resource for recreation, water supply, waste disposal and other potential uses, enabling establishment of standards of water quality. The far–reaching impact of pollution from high frequency storms occurring in the Atlanta area emphasizes the need for such a regional plan.

The Federal government has recommended maintenance of the water quality level in the South River above Snapping Shoals sufficient to support fishing. In view of the intrastate nature of the stream, the recommendation is not binding. However a point should be determined above which waste assimilation is the declared purpose of the stream. Waste treatment should be implemented to maintain whatever water quality standards are necessary to support the declared stream use below that point. Such a procedure would establish a logical framework upon which optimal expansion of waste treatment facilities may be based.

The procedures would require development of a model of the hydrology and water quality of the South River. The various studies that have been performed on the river and the detailed information concerning pollution characteristics of the Atlanta area determined in this study, should simplify development of such a model. The model would be used to assess the impact of changes in area development and waste treatment practices upon the water quality. Required reduction of pollution loads would depend upon the choice of river use below the reach utilized for waste transport. Table 25, compiled by McKee and Wolf (1963), presents a typical analysis of water quality requirements to meet various stream use objectives.

WATER QUALITY OBJECTIVES AND MINIMUM TREATMENT REQUIREMENTS

Water Quality Objectives, Applicable to Receiving Waters, for Salt and Fresh Surface Waters and Underground Waters

Water quality→ water uses ↓	Organisims of the coliform group	Floating, suspended & setticable solids & sludge deposits	Taste- or odor-producing substances	Dissolved oxygon	pH	Toxic, colored, or other deleterious substances	Phenolic compounds	Oil	High temperature wastes	Minimum treatment requirements for domestic sewage
A. WATER SUPPLY, DRINKING, CULINARY & FOOD PROCESSING Without treatment other than simple disinfection and removal of naturally present impurities	Most probable number coliform bacterial content of a representative number of samples should average less than 50 per 100 ml. in any month	None attributable to sewage, industrial wastes or other wastes or which, after reasonable dilution & mixture with receiving waters, interfere with the best use of these waters for the purpose indicated	None attributable to sewage, industrial wastes, or other wastes	Greater than five (5) parts per million except for underground waters	Hydrogen ion concentration expressed as pH should be maintained between 6.5 and 8.5	None alone or in combination with other substances or wastes in sufficient amounts or of such nature as to make receiving water unsafe or unsuitable for use indicated (U.S.P.H.S. Stds.)	Less than five (5) parts per billion	None	Not in sufficient quantities alone or in combination with other wastes to interfere with the use indicated	Sedimentation and effective disinfection
B. WATER SUPPLY, DRINKING, CULINARY & FOOD PROCESSING With treatment equal to coagulation, sedimentation, filtration, disinfection and any additional treatment necessary for removing naturally present impurities	M.P.N. coliform bacterial content when associated with domestic sewage of a representative number of samples should average less than 2000 per 100 ml. and should not exceed this number in more than 20 per cent of samples examined in any month	Same as for use "A" above	None attributable to sewage, industrial wastes, or other wastes which, after reasonable dilution & mixture, will increase the threshold odor number above eight (8)	Greater than five (5) parts per million except for underground waters	Same as for use "A" above	Same as for use "A" above	Less than five (5) parts per billion	None alone or in combination with other substances or wastes as to make receiving water unfit or unsafe for the use indicated	Same as for use "A" above	Sedimentation and effective disinfection
C. BATHING, SWIMMING AND RECREATION Note: When waters are used for recreational purposes such as fishing & boating, exclusive of bathing & swimming, the number "1000" may be substituted for "240" in statement of coliform objective	Coliform bacterial content of a representative number of samples should average less than 240 per 100 ml. and should not exceed this number in more than 20 per cent of samples examined when associated with domestic sewage (see note under "C" at left)	Same as for use "A" above	None attributable to sewage, industrial wastes, or other wastes which, after reasonable dilution & mixture, will interfere with the best use of these waters for the purpose indicated	Greater than five (5) parts per million	Same as for Use "A" above	Same as for Use "A" above	Less than 25 parts per billion or none in sufficient amounts such as to impart a residual taste to recreational or commercial fish, shellfish, or other aquatic forms	Same as for use "B" above	Same as for use "A" above	Sedimentation and effective disinfection
D. GROWTH & PROPAGATION OF FISH, SHELLFISH & OTHER AQUATIC LIFE	Coliform bacterial content of a representative number of samples should not have a median concentration greater than 70 per 100 ml. in waters used for the growth & propagation of shellfish	Same as for use "A" above	None attributable to sewage, industrial wastes, or other wastes which will interfere with the marketability or propagation of recreational or commercial fish, shellfish, or other edible aquatic forms	Greater than six (6) parts per million	Same as for use "A" above	None alone or in combination with other substances or wastes to sufficient amount or of such character as to make receiving waters unsafe or unsuitable for use indicated	Same as for use "C" above	Same as for use "B" above	None in sufficient quantity as to be injurious to or interfere with the normal propagation of fish, shellfish, or other aquatic life	Sedimentation for all uses under this group but disinfection required in addition only if discharged into waters used for the growth & propagation of shellfish, either commercial or recreational
E. AGRICULTURAL AND INDUSTRIAL WATER SUPPLY Without treatment except for the removal of natural impurities to meet special quality requirements, other than those classified under "A" above. Note: For agricultural water supply, salinity and sodium hazards are determined by electrical conductivity (EC $\times 10^6$) and sodium adsorption ratio (SAR). Waters high in both salinity and sodium are generally unsuitable for irrigation purposes. (See "classification and use of irrigation waters", circular No. 969, U.S. Department of Agriculture, November, 1955)		Same as for use "A" above	None attributable to sewage, industrial wastes, or other wastes which will adversely affect the marketability of agricultural or industrial produce	Greater than three (3) parts per million	Hydrogen ion concentration expressed as pH should be maintained between 6.0 & 9.5	Same as for use "A" above	None in sufficient quantity as to make receiving water unsuitable for use indicated	Same as for use "B" above	Same as for use "A" above	Sedimentation and effective disinfection

Table 25

SECTION XII

POLLUTION ABATEMENT APPROACHES

Alternate Solutions As Applied To Principal Pollution Load Sources

Assessment of pollution load sources demonstrates that at dry weather flow conditions, treatment plant effluents and bypasses are the most important sources of pollution load upon the stream. During storms, however, the largest fraction of the load comes from overflows and bypassed flows. Both conditions are covered by alternate abatement approaches to be discussed in the following paragraphs.

Wastewater Treatment Plants

Two different aspects must be considered in reducing pollution from wastewater treatment plants during dry weather: elimination of the need for bypassing under certain flow conditions, and general improvement of effluent quality during normal plant operation.

Modifications to Eliminate Bypassing

As discussed earlier, bypassing at existing treatment facilities is presently required when either quality of quantity of influent, or both exceeds the capabilities of the plant.

At the Intrenchment Creek wastewater treatment plant, where bypassing is more frequent, grease, grit and mud reach the plant at times in amounts that would upset the treatment units if allowed to flow through the plant. Separation of these pollutants ahead of the plant may be the only way to reduce the need for bypassing during dry weather.

Large amounts of grease reaching the treatment facilities at times, can be traced to industrial operations within the basin. Smaller amounts may originate from illegal dumping of used lubricating oils and other sources.

The City of Atlanta has established Standards of Acceptability of industrial trade waste for admission into city sewers. Regulations established to delineate these standards, if strictly enforced, should suffice to control the discharge into the sewer system of materials known to interfere with plant operation.

Construction of grease separators ahead of treatment facilities should be considered, in order to reduce the need for bypassing during dry weather.

The presence of large amounts of grit in the influent is a problem common to most existing treatment faciliites in the Atlanta area. Source of this grit has been described at the

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beginning of Section VII. Part enters the interceptors during storms through regulator devices at combined sewer overflows. Although reduction at the source is the preferred solution whenever feasible, the need for separation and removal of grit at the overflow points has long been recognized. A structure has been designed for this purpose at the Deering Road overflow, located within the Peachtree Creek basin on the north side of the City. Similar structures at the three major overflow points within the South River basin would undoubtedly reduce amounts of grit reaching the plants and consequent need for bypassing. A less expensive alternative to this solution is discussed later. It should be noted that the presence of suspended and colloidal material in storm flows is primarily due to the abundance of clays in the Atlanta area.

Modifications to Improve Effluent Quality

Operating performance under normal conditions at the treatment plants has been discussed in Section VII. Both are performing efficiently for the types of processes involved, yielding average removal of 83 percent of influent BOD. However, the impact of the treated effluents upon the dissolved oxygen level of the receiving stream is significant. The water quality may be expected to deteriorate further as wastewater flows increase in future years. Any decision to improve effluent quality would have two beneficial effects. Quality of dry weather flows in the South River would be improved, possibly permitting uses for this stream other than treated effluent disposal. If bypassing is eliminated or reduced, improved effluent quality during storms will ameliorate conditions in the receiving stream due to storm runoff as well as combined sewer overflows, should the latter remain untreated.

Such benefits should be weighed against the cost of converting existing facilities to aeration plants, which would permit BOD removal efficiencies up to approximately 95 percent. The resulting pollution load of the effluent would be less than half of the existing load and would cause a concomitant rise in water quality.

Studies concerning treatment plant modifications to alleviate pollution during dry weather flow should be undertaken in the near future.

Combined Sewer Overflows

Several alternatives may be considered for alleviating pollution of the South River by combined sewer overflows and bypassing of intercepted combined flows at the wastewater treatment plants. The feasibility, benefits and economics of each are considered in the following paragraphs.

Separation of Systems

Elimination of combined sewer overflows could be accomplished through complete separation of the sanitary and storm sewer systems within combined sewer areas. Two different aspects are involved in this separation, the within–the–right–of–way aspect or sewer system separation, and the within–the–buildings aspect, involving plumbing and connection modifications.

The first aspect, modification of the sewer system for separate conveyance of sanitary sewage and storm runoff, could be accomplished more easily in this area through construction of a new separate sanitary system and reconnection of house services to this system, leaving the existing combined sewers for storm runoff use exclusively. Since plumbing arrangements of most existing buildings combine storm and sanitary discharges into the house service, within–the–building plumbing modifications would also be required.

Disregarding momentarily considerations of difficulties to be found in actual, successful, system separation and of the amount of pollution in storm water runoff from urban areas, a cost estimation for separation has been made. It must be emphasized that in the South River area, two circumstances would favor separation, as compared to other areas of Atlanta or of most other cities with combined sewers in the Nation. First, areas served by combined sewers within the basin are located at the headwaters, and do not receive contributions of sanitary sewage from separate systems upstream. Consequently, sewers required for separation of sanitary flows can be sized exclusively for the needs of the adjacent areas to be served, and not to accomodate separate flows entering from other areas upstream.

The second favorable condition is that most of the combined sewer system lies in the Model Neighborhood area, where redevelopment is scheduled, facilitating reconnection of buildings to a new separate system.

For comparative cost estimating purposes a system of separate sanitary sewers capable of serving the area was developed on a set of the project's drainage maps for the combined sewer areas. Separate sanitary sewers were extended to the points where the existing interceptors presently begin, and were assumed to be connected to them.

Current estimating prices for new construction in the Atlanta area were utilized in calculations. These prices apply to peripheral areas, and were multiplied by a factor of 1.6 to allow for increased construction difficulties within the existing combined sewer area, with heavy traffic, tall buildings, narrow streets and an interfering, extremely complex, existing utility system in the densely developed area. Additional difficulties, such as the need to maintain uninterrupted service while reconnecting buildings and interference of existing gravity lines with new ones to be laid on grade, are very difficult to assess. Temporary pumping of sewage would be required at entire groups of buildings to permit reconnection.

In addition to within–the–right–of–way construction costs, plumbing modifications would be requried at many buildings. Internal plumbing conversions have been estimated to cost from $1,000 to $1,400 per dwelling unit on recent studies at other cities. A figure of $1,800 per dwelling unit is reasonable for an estimate of the cost of modification within private property in Atlanta. A summary of estimated costs follows for each combined sewer subbasin. A cost breakdown is included in Appendix A.

Separation of the sanitary and storm sewer systems would eliminate overflows from combined sewer areas, however, pollution resulting from runoff from storm sewered areas would remain. As shown in Section VI, such pollution is significant, ranging from one–half to three–quarters of the pollution from combined sewers. A weighted average of 55 percent appears reasonable as an estimate of this ratio. Therefore separation would reduce total annual BOD from existing combined sewer overflows from 460 lb/acre to 253 lb/acre at an estimated cost of $7,859/acre. Separation would also alleviate the need for bypassing intercepted combined flows at treatment plants, thereby further reducing pollution load upon the South River by about 690,000 lbs BOD/year. Amortization of costs over a 25–year period at 6.5 percent yields a cost–benefit ratio, expressed as dollars/lb BOD removed, of $1.82.

SUMMARY

	Within the Right–of–Way	Within Private Property	Total Cost
Area No. 1, Confederate Avenue	$ 3,706,750	$ 2,268,000	$ 5,974,750
Area No. 2, Boulevard	13,977,840	9,360,000	23,337,840
Area No. 3, McDaniel Street	4,435,260	2,970,000	7,405,260
Area No. 3A, Joyland Park	275,420	36,000	311,420
Total Costs	**$22,395,270**	**$14,634,000**	**$37,029,270**
Average Cost Per Acre	**$ 4,753**	**$ 3,106**	**$ 7,859**

Regulator Modification

As pointed out in Section V, interception and subsequent bypassing of significant fractions of the runoff from the small, high frequency storms, constitutes a major constraint on the effectiveness of overflow storage as a pollution abatement alternative. This practice also constitutes the primary source of pollution during storm flows due to the release of untreated sewage tributary to the interceptor. A most effective and feasible means of alleviating this problem would entail modification of the regulators to preclude introduction of all combined flows to the interceptors. In this way overflows could be contained in storage reservoirs for subsequent treatment, prior to discharge to the receiving stream. Furthermore, the need for bypassing raw sewage and poor quality intercepted combined flows at the treatment plants would be eliminated. Grit and grease removal facilities at the treatment plants would not be as necessary for successful operation, since detention in the reservoirs would separate much of this material.

Several feasible alternatives exist for modifying the three regulators in the study area. The least effective, yet easiest approach would be to limit the volume of intercepted flow by decreasing the size of the regulators so that only peak dry weather flows would be accepted. Greater fractions of the small combined flows that constitute the major part of the combined sewer pollution problem, would be contained in the storage reservoirs. Intercepted combined flows would not be as great at the treatment plants, thereby decreasing the necessity for bypassing. A certain amount of grit and grease would still be carried with the intercepted flow, however, storage and treatment facilities for these materials at the treatment plant would not need to be as extensive as would be required for present intercepted flows.

A second alternative would entail installation of semi–automatic, rack and pinion drop–gates at the regulator walls, activated by remote control and telemetering equipment. Such a device would be activated at the outset of rainfall in the area, averting interception of all storm flow. The gate would be raised after runoff cessation, permitting interception of normal sewage flows. Approximate cost of a reliable system of this type is estimated to be $40,000 for all three regulators.

Fully automatic devices, activated by rain gages and flow monitoring equipment in each area, could be installed at an estimated cost of $50,000. This approach would obviate human error in operation of the regulators.

Of these three alternative modifications to existing regulators, the third would be most effective. The fraction of bypassing events at the treatment plants caused by intercepted combined flows is approximately three–quarters. Based upon this estimate, installation of automatic regulators would prevent bypassing of 506,000 lbs BOD/year at Intrenchment Creek and 183,000 lbs BOD/year at South River wastewater treatment plant. With an estimated total expenditure of $50,000, the amortized cost–benefit ratio for modification of the two regulators contributing to the Intrenchment Creek interceptor would be $0.0057/lb BOD removed. The amortized cost–benefit ratio for installation of an automatic regulator at the McDaniel Street overflow would be $0.0074/lb BOD removed.

It is emphasized that only a small percentage of the flow in treatment plant bypasses originates at the combined sewer regulators, although this percentage is sufficient to cause the bypasses in most cases. Therefore prevention of bypasses by automatic closing of the regulators during storm flows will not increase significantly the amount of pollution already existing in combined sewer overflows. Maximum intercepted storm flows at Confederate Avenue and Boulevard are approximately 14 and 7 cfs respectively, whereas overflows may reach several hundred cfs.

In–System Storage

A study of possible in–system storage in the South River basin is discouraging. Slopes of the order of 0.5 percent to 1 percent in the large combined sewers of the area introduce definite volume limitations. Furthermore, age, condition and construction materials of many sewer sections weigh against extended use of in–system storage.

Off–System Storage

Storage at or near the overflow points offers a definite and economical possibility in the study area. Maximum possible storage areas utilizing undeveloped low land adjacent to the stream beds are shown in Figures 45 and 47. Figure 46 shows the area covered by containment of water to elevation 934 feet, whereas Figure 48 shows the area to 878 feet. Curves on Figures 45 and 47 are storage volumes at different surface elevations in both areas based on present topography. One of these low areas, at the McDaniel overflow, could store overflow volumes corresponding to 100 percent of the BOD from a one–year storm by utilizing all land up to elevation 931 feet.

Storage would facilitate treatment of overflows, and subsequent disposal at a reduced rate of flow. Due to the larger flow volumes, smaller percentages of overflow BOD can be stored at existing undeveloped land at the Confederate Avenue and Boulevard overflows. Sanitary land fills developed in the past in the low areas adjacent to the stream have set limitations to available storage volumes. However, storage of 100 percent of the BOD from all six–month storms is still possible within present flood plains up to elevation 879 feet.

Development of treatment alternatives for stored overflow from combined sewers requires a decision concerning the size of the "design storm" to be handled by the system.

Many published studies of combined sewer pollution abatement schemes have designed storage and treatment facilities sufficient to handle storms of low frequency. Typical design storm recurrence intervals are 10 and 15 years. While consistent with present drainage practice, this assumption is unnecessarily conservative for alleviation of pollution from combined sewers. Results of this study indicate that two–week and higher frequency storms yield 57 percent of the total annual BOD discharged to the South River from combined sewers. Whenever feasible, provision for storage and treatment of all flows from these small storms will also enable storage and treatment of significant fractions of flows from larger storms, thereby permitting treatment of at least three–quarters of the total annual BOD. It will also alleviate the considerable shock impact of overflows from the individual high–frequency storms. The storm of two–week average recurrence interval is therefore recommended as a reasonable "design storm" upon which to base plans for alleviation of the effects of pollution from combined sewer overflows.

Storage required at the Confederate Avenue and Boulevard overflows will be 1.8 million cubic feet, which from Figure 48 entails submergence of 10.7 acres of land up to elevation 873 feet. Required storage at the McDaniel Street reservoir would be 0.5 million cubic feet,

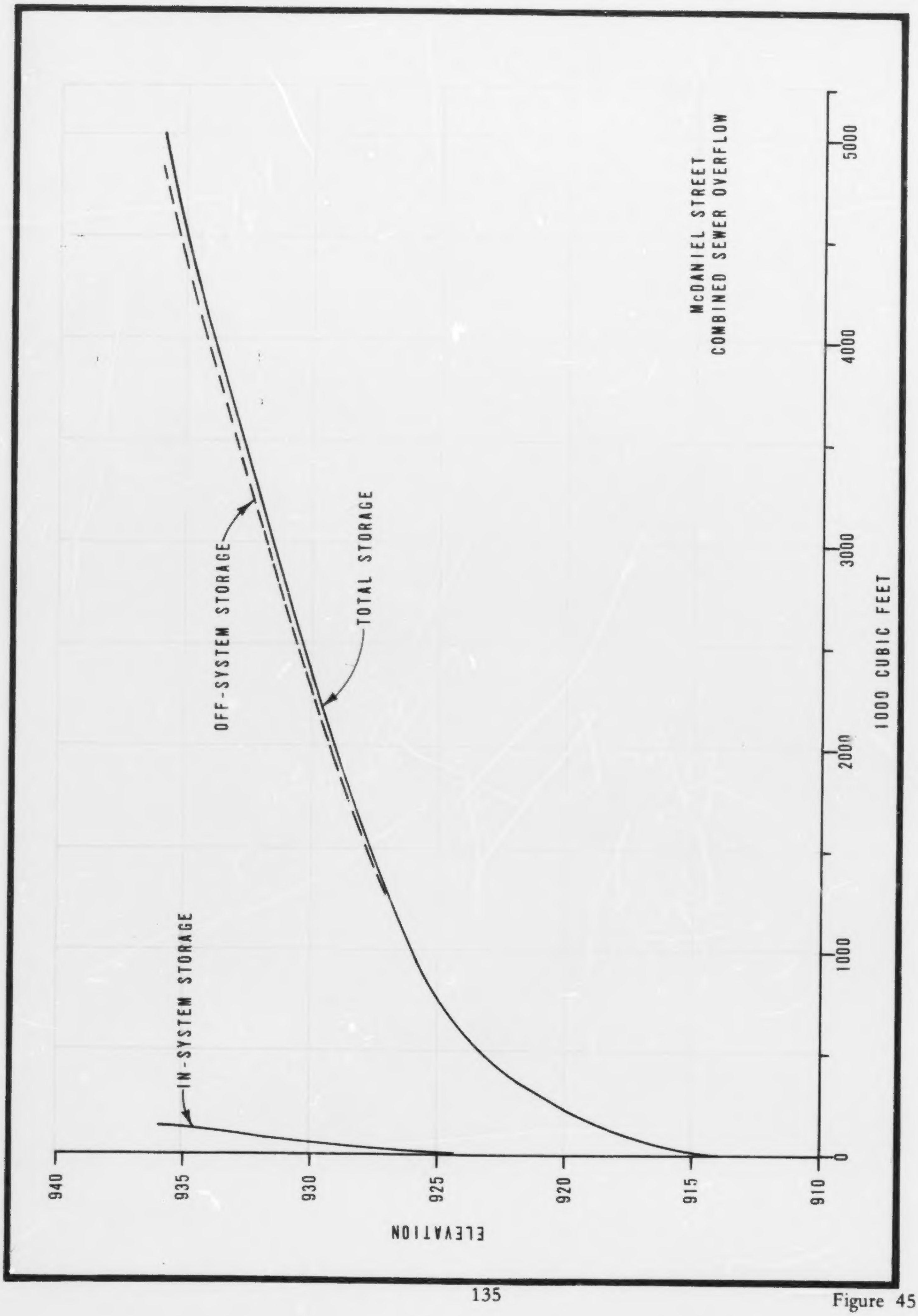
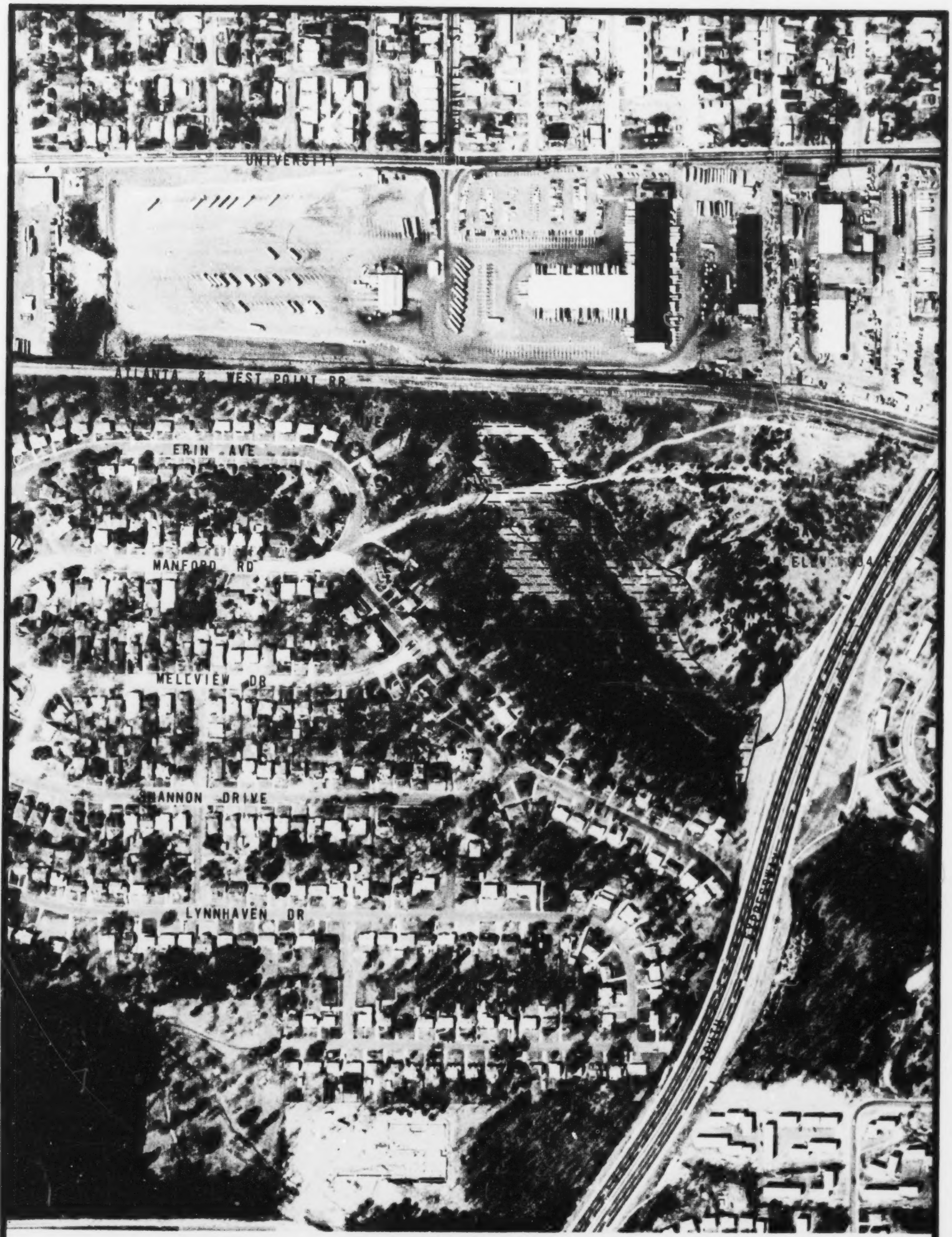

McDANIEL STREET
COMBINED SEWER OVERFLOW
IN-SYSTEM STORAGE
OFF-SYSTEM STORAGE
TOTAL STORAGE
ELEVATION
910
915
920
925
930
935
940
1000 CUBIC FEET
0
1000
2000
3000
4000
5000

Figure 45


UNIVERSITY AVE
ATLANTA & WEST POINT RR
ERIN AVE
MANFORD RD
MELLVIEW DR
SHANNON DRIVE
LYNNHAVEN DR
ELEV 934
SOUTH EXPRESSWAY

STORAGE AREA
McDANIEL ST
OVERFLOW

Figure 46

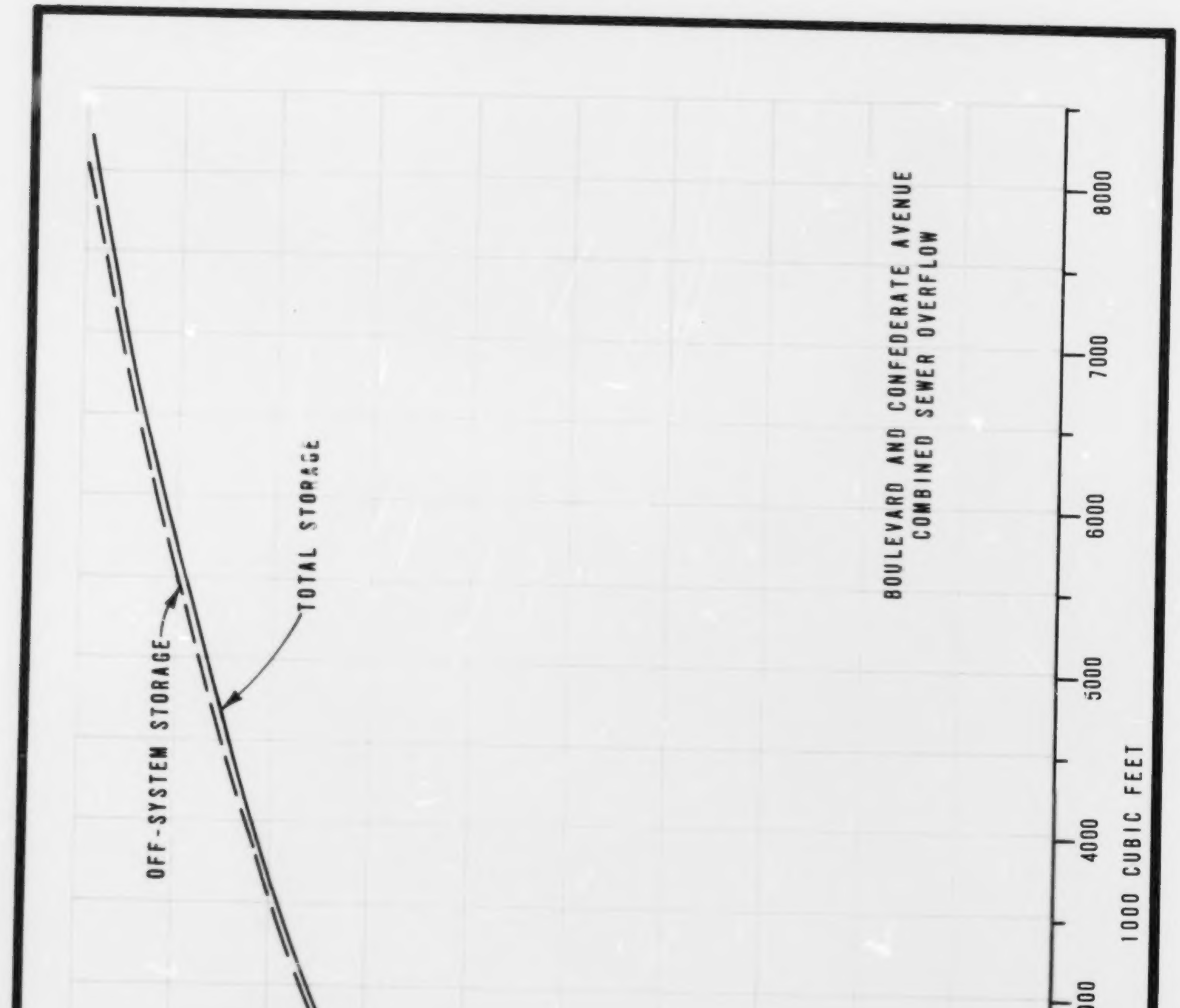

BOULEVARD AND CONFEDERATE AVENUE
COMBINED SEWER OVERFLOW
IN-SYSTEM STORAGE
OFF-SYSTEM STORAGE
TOTAL STORAGE
ELEVATION
880
878
876
874
872
870
868
866
864
862
860
0
1000
2000
3000
4000
5000
6000
7000
8000
1000 CUBIC FEET

Figure 47

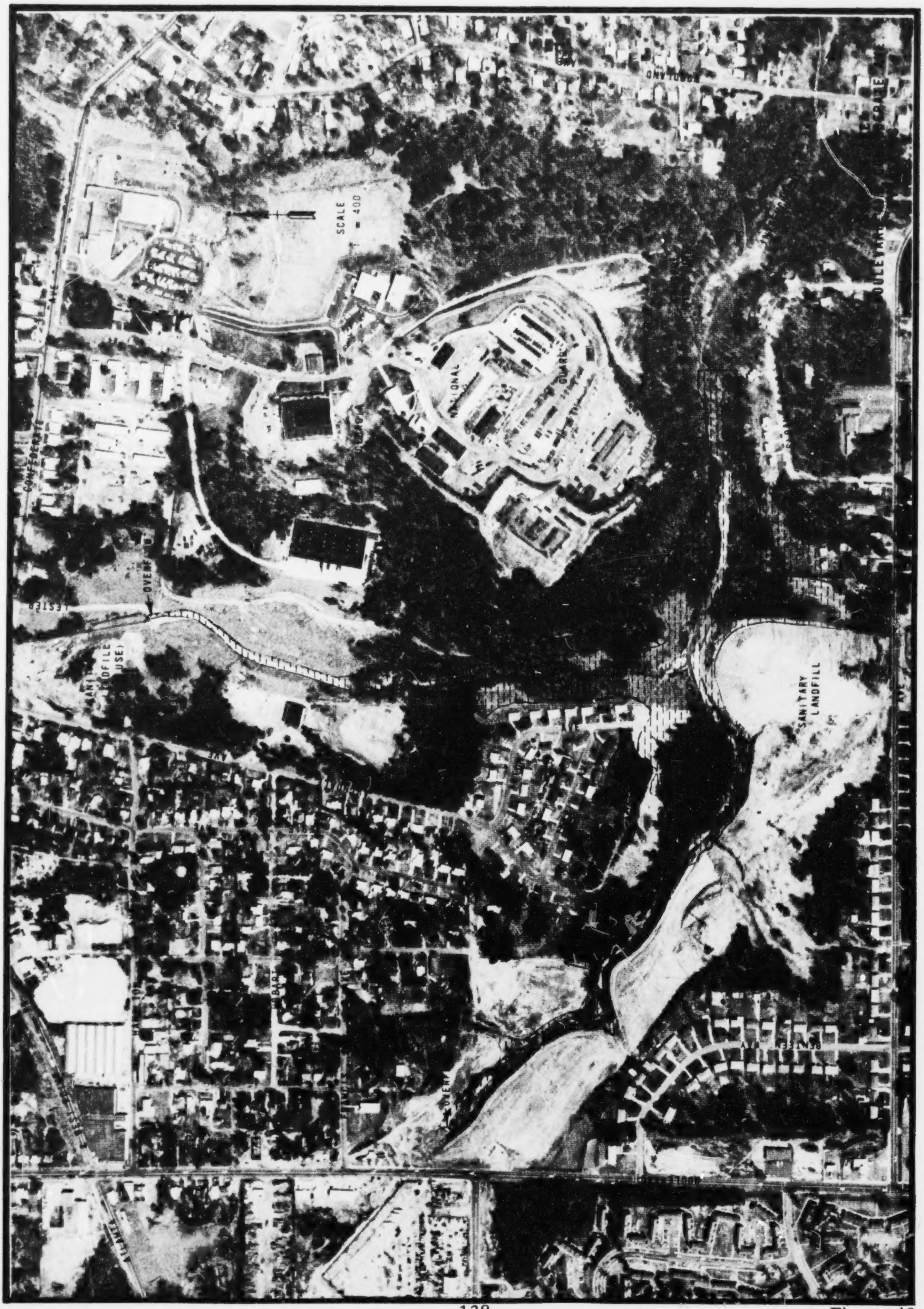

SCALE
= 400
NATIONAL
SANITARY LANDFILL
BOULEVARD
CONFEDERATE
LESTER
HOBART

Figure 48

necessitating submergence of 2.9 acres up to elevation 923 feet. These elevations are based upon existing topography, whereas it would be necessary to improve the ground prior to utilization as a reservoir.

A primary advantage of overflow storage is the modulation of peak flow rates. In general, the larger reservoir yields the greater modulation. The number and total capacity of treatment units need only meet the modulated peak flow rate. For those storms entirely contained in the reservoir, as is the case for the "design storm," a major constraint on treatment capacity is the desirability of rapidly providing storage for subsequent overflows. Furthermore, rapid emptying of the reservoir will prevent problems that may occur due to depletion of oxygen resources. Whatever the detention time, a certain amount of solids will settle, both organic and inorganic. Due to the organic nature of part of the solids, and the open reservoir employed for storage, solids should be removed from the reservoir.

Successful removal of solids has been reported at similar installation in Chippewa Falls, Wisconsin (Banister, 1969), utilizing a street sweeper. Removal by automatic mechanical equipment would be desirable, however, the irregular shapes and large surface areas of the reservoirs would require non–standard equipment at considerable expense.

Construction costs for the reservoir and dam at the Confederate Avenue and Boulevard overflow sites on Intrenchment Creek are estimated at $600,000. This figure includes land at $15,000/acre. Combined sewer overflows are assumed to fill the reservoir and overtop the dam under conditions exceeding design capacities. Costs exclude handling and treatment expenses, which are considered subsequently. A similar construction estimate for the McDaniel Street overflow storage site is $125,000. A dam at this site is unnecessary since the downstream end of the reservoir abuts against a considerable volume of fill underlying Interstate 85 (South Expressway). An overflow weir and existing culvert may be used to drain the reservoir after a storm.

Once the combined sewer overflows are stored, a number of alternatives are available for treatment and release to the receiving streams. The feasibility, economics and benefits of each are considered in the following paragraphs.

Alternate A

The first approach comprises storage and solids removal in the reservoir followed by physical treatment of the overflows and release. The two–week design storm of five–hour duration may be expected to yield 13.1 million gallons of combined sewer overflow to the Intrenchment Creek reservoir and 3.6 million gallons to the McDaniel Street reservoir. Lesser volumes would be expected from shorter duration storms, as shown in Figures 28 and 29.

In order to obtain full benefit of the reservoirs, they should be emptied as soon as possible after cessation of runoff. The requisite high treatment capacities increase costs significantly. However, the higher capacities also permit treatment of greater fractions of overflows from larger storms, and reduce the need for aerating stored overflows. Table 26 presents the percentage of 1969 overflow volume that would have been treated in the Intrenchment Creek reservoir if designed to contain a two–week storm. Options A, B and C represent 2–, 5– and 10–hour durations from cessation of overflows to reservoir emptying, assuming treatment begins at commencement of overflows.

TABLE 26

Storm		Percent Overflow Volume Treated		
Recurrence Interval	Duration (hours)	Option A	Option B	Option C
2 week	1	100	100	100
	2	100	100	100
	5	100	100	100
4 week	1	100	100	100
	2	100	97	90
	5	100	100	100
	10	100	100	100
6 month	2	33	31	30
	5	42	38	35
	10	42	37	38
Required Treatment Capacity (mgd)		**35**	**26**	**19**

Option A enables treatment of about 88 percent of the total annual overflow volume passing through the reservoir. Options B and C will enable treatment of a slightly lower percentage, due to the infrequent occurrence of storms separated by time intervals too short to permit complete draining of the reservoir. An estimate of 80 percent treatment with Option C appears reasonable.

Overflows are initially saturated with dissolved oxygen. Unless they are released at a reasonable level of DO, impact upon the receiving stream of the treated overflows will be deleterious due to the decrease in streamflow after the storm. Under average conditions it is estimated that the stored volume may become anaerobic within several hours of overflow cessation. However further lab work is required to confirm this estimate, including determination of the deoxygenation rate constant. In order to maintain effluent from the reservoir at a DO of at least five mg/l, prechlorination or aeration will probably be necessary for Option C. Option B may require similar facilites, but these should be unnecessary for Option A.

Although further tests are required to confirm the estimate, prechlorination would entail application of an estimated 30 mg/l to the reservoir influent. Preliminary investigation of this approach indicated that due to the high flow rates encountered, prechlorination would be infeasible. The large drainage areas tributary to the reservoirs cause high flow rates from even small storms. Diffused and mechanical surface aeration are also considered infeasible, due to the high velocities that may occur and the low transfer efficiencies encountered at high DO levels. However, introduction of oxygen to the effluent conduit in either diffused or dissolved form should enable maintenance of high DO levels. The partial pressure driving force will be maximized by introduction of oxygen at this point and flow rates will be steady and relatively low. A downstream monitor would enable automatic adjustment of oxygen feed rate to meet the required DO level. Further lab studies and detailed information concerning the oxygenation process will enable assessment of the optimal procedure and expected annual operating cost.

Treatment of combined sewer overflows is best accomplished by physical rather than biological means, due to the short detention times generally possible. Development of reliable cost estimates for large scale facilities is hampered by the lack of such facilities to date. Two basic processes are considered: screening and flotation. However, several variations reported in the literature indicate that a major factor in costs and efficiencies of these processes is the screen mesh size.

Microstraining of overflows has been proposed as an economically competitive alternate to separation of combined sewers (Keilbaugh, Glover and Yatsuk, 1969). Utilizing a screen mesh of 23 microns, preceded by a heavy solids trap and bar screen, this process achieved removal of suspended solids averaging 80 to 90 percent, and volatile solids averaging 70 percent. However, BOD removal is unpredictable. It is possible that the large surface area of small particles leaving the microstrainer enhances bacterial action. Such an effect would increase efficiency of effluent chlorination. If chlorination is not implemented, however, the shock impact of the treated flows upon the South River might be substantial. Based upon an 11–acre study area without overflow storage, costs of microstraining and chlorination were estimated at $10,500 to $12,800 per acre served. Expansion of these costs to an area of 3,400 acres is of little value, however rough estimates of this process suggest that it may be implemented for 40 to 50 percent of separation costs.

Fine screening of overflows with a 105 micron, rotary, vibratory screen has been reported to remove 99 percent of floatable and settleable solids, 34 percent of total suspended solids, and 27 percent of COD (Cornell, Howland, Hayes and Merryfield, 1970). BOD reduction was not calculated. Estimated cost of a 25 mgd screening facility is $538,000. Annual operation and maintenance costs are estimated at $18,500. Total annual cost, including debt service, is $59,500, based upon a design life of 25 years with amortization at 6.5 percent.

Mason (1969) studied the possibility of combining screening and dissolved air flotation. Using a 297 micron rotary screen preceded by a 1/2 inch bar rack and excluding flotation, this process achieved a 22 percent BOD reduction, 28 percent COD reduction, 27 percent suspended solids reduction and 26 percent volatile suspended solids reduction. Addition of flotation with chemical flocculation increased removal efficiencies for BOD and COD to 51 and 53 percent, respectively. TSS and VSS removals averaged 68 and 65 percent. Capital costs were estimated at $5,000 to $8,000/mgd for plants exceeding 50 mgd. Operating costs should approximate 4.5 cents/1,000 gallons.

The same combination of processes was studied by Rhodes Technology Corporation (1970) utilizing a 32–mesh gyratory screen followed by a series of hydrocyclones. Alum and a polyelectrolyte were added to the influent of the flotation tank. Results indicated that this process will remove 84 percent of the suspended solids passing the screen, and 42 percent of the BOD. At capacities greater than 8 mgd, operating and maintenance costs increase substantially such that whenever land is available, primary clarification is less expensive. In this application the land is not available; moreover, the reservoir will act as a primary clarifier to some extent. Total annual cost of treatment for options A, B and C with this approach, assuming 25 year bonds at 6.5 percent, is estimated at $75,000, $57,000 and $42,000, respectively, at Intrenchment Creek. Similar estimates for the McDaniel reservoir would be $26,000, $20,000 and $16,000. All estimates exclude oxygenation costs, which cannot be determined at this time.

Certain factors are pertinent to any treatment processes at the reservoir sites. Detention time in the reservoirs will induce settling of solids, although settling tests on composite samples of combined sewer overflows are necessary to determine percent removal of solids and BOD. It is estimated that 20 percent BOD removal and 60 percent solids removal may be possible within the reservoirs due to settling. Solids removed by treatment at Intrenchment Creek may be discharged to an interceptor along the north bank of the reservoir site. Similarly solids removed by the McDaniel treatment unit may be discharged to an existing sewer at the site.

Consideration of the four treatment alternatives is complicated by the lack of comparability of cost information. However, for a 25 mgd facility, the cost of microstraining would be very high and BOD removal unpredictable. Fine screening alone would achieve 27 percent COD reduction for a total annual cost of $59,500. Coarse screening and dissolved air flotation would achieve 42 percent BOD reduction for a total annual cost of $55,000. It must be emphasized that these costs are rough estimates only, and are not strictly comparable. However, they indicate the approximate efficiency and cost of existing treatment alternatives for combined sewer overflows. Consequently the last approach is selected for subsequent benefit–cost calculations.

The total cost of alternative A is shown in Table 27 for the three reservoir detention times. It is apparent from the Table that the least costly option, the 10–hour detention time, has the most favorable cost–benefit ratio. It may therefore be concluded that storage of combined sewer overflows in a reservoir with capacity to contain all two–week storms, followed by screening, dissolved air flotation and chlorination at a capacity designed to empty the reservoir within 10 hours of overflow cessation, will enable removal of approximately 53 percent of treated overflow BOD and 87 percent of suspended solids, at a cost of about $0.17/lb BOD.

TABLE 27

Reservoir Site	Detention Time After Cessation of Overflow (hours)		
	2	5	10
Intrenchment Creek Reservoir			
Capital Cost ($)	600,000	600,000	600,000
Amortized Annual Cost ($)	49,000	49,000	49,000
Cleanup Annual Cost ($)	2,000	2,000	2,000
Treatment Annual Cost ($)	75,000	57,000	42,000
Chlorination Annual Cost ($)	29,000	29,000	28,000
Total Annual Cost ($) (excluding oxygenation)	155,000	137,000	121,000
Annual Flow Treated (%)	88	85	80
BOD Removal Efficiency (%)	53	53	53
BOD Removal (lbs/year)	762,000	736,000	692,000
Cost–Benefit Ratio ($/lb BOD)	0.20	0.19	0.17
McDaniel Street Reservoir			
Capital Cost ($)	125,000	125,000	125,000
Amortized Annual Cost ($)	10,000	10,000	10,000
Cleanup Annual Cost ($)	1,000	1,000	1,000
Treatment Annual Cost ($)	26,000	20,000	16,000
Chlorination Annual Cost	10,000	10,000	9,000
Total Annual Cost ($) (excluding oxygenation)	47,000	41,000	36,000
Annual Flow Treated (%)	88	85	80
BOD Removal Efficiency (%)	53	53	53
BOD Removal (lbs/year)	208,000	201,000	189,000
Cost–Benefit Ratio ($/lb BOD)	0.22	0.20	0.19

NOTE: Amortization over 25 years at 6.5 percent

Alternate B

A possible alternative to treatment of stored overflows at the reservoir sites is treatment at the existing treatment plants. Retention of overflows in the reservoirs for a few hours will permit grit and some solids to settle. Release of flow to the existing sewer lines can be remotely controlled according to sewage flows at the treatment plants. Peak hydraulic capacities of the plants can be utilized at night when sewage flows are low, and at times during the day when flows are less than peak ratings.

Design capacities at the Intrenchment Creek and South River wastewater treatment plants are 20 and 18 mgd respectively. At these rates BOD removal is about 83 percent at both plants. Hydraulic overloading will cause BOD removal efficiency to drop, such that flows at 250 percent of design capacity may yield approximately 50 percent BOD removal. Additional clarification units would permit higher treatment of overflows at these flow rates.

With the Intrenchment Creek wastewater treatment plant operating at design capacity on dry weather sewage flow, a surcharge of combined sewer overflow yielding a steady hydraulic rate through the plant of 47 mgd would permit emptying of the Intrenchment Creek reservoir in 12 hours. During this period it is estimated that BOD removal within the plant would average 50 percent. Expansion of facilities would permit higher BOD removal at this flow rate. Additional studies would enable determination of whether the 12–hour detention time might be extended, permitting lower hydraulic loading rates.

When design capacity of the South River wastewater treatment plant is reached, a surcharge of storm overflow yielding a steady flow rate of 25 mgd will permit emptying of the McDaniel Street reservoir in 12 hours. BOD removal efficiency would fall to about 73 percent.

Although this approach would enable treatment of combined sewer overflows, it will reduce the treatment efficiency for sewage entering the plants during the 12 hours of peak flows. It is estimated that net BOD reduction at the Intrenchment Creek reservoir and wastewater treatment plant will be –423,000 lbs/year, indicating that the loss in treatment efficiency offsets any advantage gained by treating the overflow. The net BOD reduction at the South River treatment plant is 48,000 lbs/year.

This approach is therefore unrealistic for treatment of flows from Intrenchment Creek reservoir. Application to flows from the McDaniel Street reservoir would yield a cost–benefit ratio of approximately $0.23/lb BOD.

Alternate C

BOD reduction due to storage, settling of solids and chlorination constitutes an additional pollution abatement alternative. As mentioned previously, further tests are necessary to determine settling and BOD removal characteristics; however, 20 percent BOD removal is a reasonable estimate. Release of stored overflows to the receiving stream may necessitate oxygenation of the effluent to maintain high DO levels. Although it is probable that decreased detention times will reduce the need for oxygenation at the expense of a reduction in BOD removal, the nature of the trade–off cannot be determined at this time.

Implementation of this approach at the Intrenchment Creek reservoir site should enable a reduction of BOD in all flows passing through the reservoir, since even flows overtopping the dam will deposit some solids while progressing through the reservoir. However BOD reduction under such unsteady conditions will probably not exceed 10 percent. Since 57 percent of the total annual BOD from combined sewer overflows may be contained in a reservoir designed to store a two–week storm, it may be assumed that the remaining 43 percent will receive 10 percent BOD removal. Consequently at this site, an estimated 256,000 lbs BOD/year may be removed. The resulting cost–benefit ratio, excluding costs of oxygenation, is $0.31/lb BOD.

The McDaniel Street reservoir, operated in the same fashion, would remove an estimated 70,000 lbs BOD/year. Excluding oxygenation costs, the cost–benefit ratio would be $0.29/lb BOD removed.

Evaluation of Alternative Pollution Abatement Schemes

Although other approaches to the abatement of pollution from combined sewers have been discussed in the literature, those discussed previously in this section are considered most applicable to circumstances in the City of Atlanta. The various alternatives are listed in Table 28 along with their cost–benefit ratios and percentage removal of total annual BOD from combined sewer overflows and intercepted flows. The latter quantity is estimated at 2,767,000 pounds.

The table reveals the desirability of regulator modification, which will reduce total annual BOD released to the South River due to combined sewers by 25 percent at a total estimated capital cost of $50,000. Storage and treatment of overflows at both reservoir sites by screening, dissolved air flotation and chlorination will increase to 57 percent the removal of total annual BOD attributable to combined sewers, at an annual cost of $165,000. However, 53 percent may be removed for $140,000/year by omitting treatment of overflows stored at the McDaniel Street reservoir. The cost–benefit ratio of the latter approach is only slightly higher than that of the former.

TABLE 28

Alternative	Cost–Benefit Ratio ($/lb BOD)	BOD Removed (Percent)
Regulator Modification (IC)	0.0057	18
Regulator Modification (M)	0.0074	7
Treatment Alt. A (IC)	0.17	25
Treatment Alt. A (M)	0.19	7
Treatment Alt. B (ICTP)	––	––
Treatment Alt. B (SRTP)	0.23	2
Treatment Alt. C (IC)	0.31	9
Treatment Alt. C (M)	0.29	3
Sewer Separation	1.82	60

NOTE:
IC – Intrenchment Creek Reservoir
ICTP – Intrenchment Creek Wastewater Treatment Plant
M – McDaniel Street Reservoir
SRTP – South River Wastewater Treatment Plant

It is emphasized that the costs and efficiencies utilized in this section involve a significant amount of engineering judgment, necessitated by the uniqueness of the proposed solutions as well as the lack of detailed information available in the literature. However, reasonable estimates of costs and efficiencies of alternate schemes have been made. Based upon these estimates, the optimal solution includes modification of regulators at the overflow points, construction of two reservoirs to store overflows from combined sewers, and treatment of the stored volumes with screening and dissolved air flotation. At an estimated total annual cost of $165,000, this solution will remove 57 percent of the annual BOD released to the South River due to combined sewer overflows and intercepted flows. It will also alleviate the considerable shock impact of flows from small, high frequency storms.

Abatement of Pollution From Storm–Sewered Areas

Consideration was given to ways of reducing the pollution load due to storm runoff in areas served by separate sewers. Compared to that from combined sewer areas, the pollution averages approximately 55 percent of the total annual BOD/acre. However, the relatively large acreage served by storm sewers yields considerably greater annual BOD. A rough estimate is that total annual BOD from combined sewer areas constitutes 30 to 40 percent of the total annual BOD from that part of the City of Atlanta contained in the study area and tributary to station 8 at Bouldercrest Road.

The study area is characterized by a general lack of suitable storage sites for the large flow volumes yielded by even small storms. Without storage, treatment must be geared to handle these volumes at very high flow rates. Preliminary investigations of treatment alternatives for combined sewer overflows under such conditions revealed the present infeasibility of this approach. The high costs of treatment, the low efficiencies of BOD removal and the relatively low waste strength combined to indicate that treatment of storm runoff is not feasible in the study area at this time.

SECTION XIII

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SECTION XIV

ACKNOWLEDGEMENTS

Grateful appreciation is due the City of Atlanta Department of Public Works, Water Pollution Control Division, and the Georgia State Water Quality Control Board for their assistance during this study.

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SECTION XV

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SECTION XVI

APPENDICES

APPENDIX A

COST ESTIMATE – SYSTEM SEPARATION

(Based on construction of separate sanitary sewers and reconnection)

AREA NO. 1 – CONFEDERATE AVENUE

A. Within the Public Right–of–Way Cost

Item	Description	Estimated Quantity	Unit	Unit Price	Amount
1.	8" Pipe	33,050	l.f.	$ 4.00	$ 132,200.00
2.	10" Pipe	1,800	l.f.	5.00	9,000.00
3.	12" Pipe	1,700	l.f.	6.00	10,200.00
4.	15" Pipe	2,100	l.f.	7.00	14,700.00
5.	18" Pipe	2,200	l.f.	9.00	19,800.00
6.	24" Pipe	3,750	l.f.	13.00	48,750.00
7.	30" Pipe	4,350	l.f.	17.00	73,950.00
8.	Earth Excavation	115,780	c.y.	4.00	463,120.00
9.	Solid Rock Excavation	11,580	c.y.	16.00	185,280.00
10.	Semi–decomposed Rock	23,160	c.y.	10.00	231,600.00
11.	Manholes	320	each	500.00	160,000.00
12.	Pavement Replaced	42,000	sq. yd.	10.00	420,000.00
13.	Reconnect Services	1,260	each	100.00	126,000.00
14.	Railroad and Highway Crossings	(8)		---	36,000.00
					$1,930,600.00

Add 60% for work in highly developed areas with interfering utilities and uninterrupted service requirement	1,158,360.00
Estimated Construction Cost	3,088.960.00
Engineering contingencies, etc. (20%)	617,790.00
Cost within the right–of–way	$3,706,750.00

B. Within Private Property Cost

1.	Modification of building, plumbing and service connections	1,260	each	$1,800.00	$2,268,000.00

Total Cost of Separation, Area No. 1	**$5,974,750.00**

Cost per acre, Area No. 1

A.	Within the public right–of–way	$ 3,283.00
B.	Within private property	$ 2,009.00
	Total cost per acre	**$ 5,292.00**

AREA NO. 2 – BOULEVARD

A. Within the Public Right–of–Way Cost

Item	Description	Estimated Quantity	Unit	Unit Price	Amount
1.	8" Pipe	136,100	l.f.	$ 4.00	$ 544,400.00
2.	10" Pipe	11,475	l.f.	5.00	57,375.00
3.	12" Pipe	12,100	l.f.	6.00	72,600.00
4.	15" Pipe	6,475	l.f.	7.00	45,325.00
5.	18" Pipe	5,175	l.f.	9.00	46,575.00
6.	24" Pipe	6,725	l.f.	13.00	87,425.00
7.	30" Pipe	3,350	l.f.	17.00	56.950.00
8.	36" Pipe	2,250	l.f.	22.00	49,500.00
9.	42" Pipe	3,325	l.f.	27.00	89,775.00
10.	48" Pipe	2,400	l.f.	33.00	79,200.00
11.	Earth Excavation	468,475	c.y.	4.00	1,873,900.00
12.	Rock Excavation	46,850	c.y.	16.00	749,600.00
13.	Semi–decomposed Rock	93,700	c.y.	10.00	937,000.00
14.	Manholes	1,300	each	500.00	650,000.00
15.	Pavement replaced	126,250	sq. yd.	10.00	1,262,500.00
16.	Reconnect services	5,200	each	100.00	520,000.00

Item	Description	Estimated Quantity	Unit	Unit Price	Amount
17.	Railroad and Highway crossings	(23)	L.S.	——	$ 158,000.00
					$7,280,125.00
	Add 60% for work in highly developed areas, interfering utilities and uninterrupted service requirement				4,368,075.00
	Estimated Construction Cost				11,648,200.00
	Engineering contingencies, etc. (20%)				2,329,640.00
	Cost, within the right–of–way				$13,977,840.00
	B. Within Private Property Cost				
1.	Modification of building, plumbing and service connections	5,200	each	$1,800.00	$9,360,000.00
	Total Cost of Separation, Area No. 2				23,337,840.00

Cost per acre, Area No. 2

A.	Within the public right–of–way	5,773.00
B.	Within private property	3,865.00
	Total cost per acre	**$ 9,638.00**

AREA NO. 3 – MCDANIEL STREET

A. Within the Public Right–of–Way Cost

Item	Description	Estimated Quantity	Unit	Unit Price	Amount
1.	8" Pipe	42,550	l.f.	$ 4.00	$ 170,200.00
2.	10" Pipe	3,800	l.f.	5.00	19,000.00
3.	12" Pipe	3,250	l.f.	6.00	19,500.00
4.	15" Pipe	2,300	l.f.	7.00	16,100.00
5.	18" Pipe	2,475	l.f.	9.00	22,275.00

Item	Description	Estimated Quantity	Unit	Unit Price	Amount
6.	21" Pipe	1,700	l.f.	10.50	17,850.00
7.	24" Pipe	1,925	l.f.	13.00	25,025.00
8.	Earth Excavation	141,520	c.y.	4.00	566,080.00
9.	Rock Excavation	9,900	c.y.	16.00	158,400.00
10.	Semi–decomposed Rock	24,060	c.y.	10.00	240,600.00
11.	Manholes	380	each	500.00	190,000.00
12.	Pavement replaced	52,000	sq. yd.	10.00	520,000.00
13.	Reconnect service	1,650	each	100.00	165,000.00
14.	Railroad and Highway Crossings	(45)	L.S.	---	180,000.00
					$ 2,310,030.00
Add 60% for work in highly developed areas, with interfering utilities and uninterrupted service requirements					1,386,020.00
Estimated Construction Cost					3,696,050.00
Engineering contingencies, etc. (20%)					739,210.00
Cost within the right–of–way					$ 4,435,260.00
	B. Within Private Property Cost				
1.	Modification of building, plumbing, and service connections	1,650	each	$1,800.00	$ 2,970,000.00
	Total cost of separation, Area No. 3				$ 7,405,260.00
	Cost per acre, Area No. 3				
	A. Within the public right–of–way				$ 4,584.00
	B. Within private property				3,069.00
	Total cost per acre				**$ 7,653.00**

AREA NO. 3A – JOYLAND PARK AREA (Smaller combined sewer area located east of I–75 at University Avenue Exit)

A. Within the Public Right–of–Way Cost

Item	Description	Estimated Quantity	Unit	Unit Price	Amount
1.	8" Pipe	2,825	l.f.	$ 4.00	$ 11,300.00
2.	10" Pipe	1,750	l.f.	5.00	8,750.00
3.	Earth Excavation	10,675	c.y.	4.00	42,700.00
4.	Solid Rock Excavation	1,075	c.y.	16.00	17,200.00
5.	Semi–decomposed Rock	2,150	c.y.	10.00	21,500.00
6.	Manholes	22	each	500.00	11,000.00
7.	Pavement Replaced	1,100	sq. yd.	10.00	11,000.00
8.	Reconnect Service	20	each	100.00	2,000.00
9.	Railroad and Highway Crossings	(3)	L.S.	---	18,000.00
					$ 143,450.00
Add 60% for work conditions interfering utilities and uninterrupted service requirement					$ 86,070.00
		Estimated Construction Cost			229,520.00
Engineering contingencies, etc. (20%)					45,900.00
Cost within the right–of–way					$ 275,420.00

B. Within Private Property Cost

Item	Description	Estimated Quantity	Unit	Unit Price	Amount
1.	Modification of building, plumbing, and service connections	20	each	$1,800.00	$ 36,000.00
	Total cost of separation, Area No. 3A				$ 311,420.00

Cost per acre, Area No. 3A

A.	Within the public right–of–way	$ 1,420.00
B.	Within private property	185.00
	Total cost per acre	$ 1,605.00

SUMMARY

	Within the right–of–way	Within Private Property	Total Cost
Area No. 1, Confederate Avenue	$ 3,706,750	$ 2,268,000	$ 5,974,750
Area No. 2, Boulevard	13,977,840	9,360,000	23,337,840
Area No. 3, McDaniel Street	4,435,260	2,970,000	7,405,260
Area No. 3A, Joyland Park	275,420	36,000	311,420
Total Costs	**$22,395,270**	**$14,634,000**	**$37,029,270**
Average cost/acre	**$ 4,753**	**$ 3,106**	**$ 7,859**

APPENDIX B

UNIT HYDROGRAPH

PROGRAM PACKAGE

This program computes the unit hydrograph using the Wiener–Hopf equations and the MPS linear programming routine on the IBM 360. At the present time this is the best way of applying a linear formulation to nonlinear data. Due to the nonlinear characteristics of urban watersheds, different unit hydrographs will be obtained for different storms in the same drainage basin. There is no valid way of averaging them.

Following is a summary of the input required for HYDRO:

		Format
a.	the storm identification number	I 6
b.	length of the rainfall record, N	I 3
c.	length of the runoff record, M	I 3
d.	rainfall record (F(I), I=1,. . .N)	20F4.2
e.	runoff record (G(I), I=1,. . .M)	10F8.2

HYDRO receives as input only one storm record at a time. The last card in the data deck must be the storm number 000000, in I 6 format, or a blank card. Data is inserted at the end of stage A, after the card //GO.SYSIN DD *.

Time increments are of arbitrary length, but must be the same for both rainfall and runoff. Both records must start at the same time, even though runoff readings may initially be zero. Array dimensions should be checked to ensure adequate machine storage allocation.

In stage A the program calculates the autocorrelation function, PHIFF, and the cross–correlation function, PHIFG, and stores the output in the computer data cell in a format suitable for input to stage B, the MPS linear programming routine. Immediately after execution of stage A, control cards for stage B are read in, followed by the data stored in the data cell. The unit hydrograph coordinates are contained in the output of stage B.

Pertinent results are listed under the heading "SECTION 2 – COLUMNS." For instance, H101 and H102 are the labels attached to the first and second unit hydrograph coordinates. The values of these coordinates are shown under the column "ACTIVITY." In all but the simplest storms, some coordinates will have zero value. This is a direct result of the method of analysis, which attempts to constrain unit hydrograph coordinates to be nonnegative.

Reference: Eagleson, P.S., Mejia, R., and March, F., "Computation of Optimum Realizable Unit Hydrographs." *Water Resources Research, 2,* No. 4, 1966 (pp. 755–764)

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FORTRAN IV G LEVEL 1, MOD 3            MAIN            DATE = 70084          09/48/03            PAGE 0001

              C       PROGRAM TO CALCULATE UNIT HYDROGRAPH USING WIENER HOPF EQUATIONS
              C       AND MPS LINEAR PROGRAMMING ROUTINE ON IBM 360
 0001                 COMMON F(45),G(80),PHIFF(45),PHIFG(80)
 0002                 DIMENSION A(120,80),MS(120)
 0003               1 READ(5,2)NSTORM
 0004               2 FORMAT(I6)
 0005                 IF(NSTORM)120,120,3
 0006               3 READ(5,5)N,M
 0007               5 FORMAT(I3,5X,I3)
 0008                 READ(5,10)(F(I),I=1,N)
 0009              10 FORMAT(20F4.2)
 0010                 READ(5,15)(G(I),I=1,M)
 0011              15 FORMAT(10F8.2)
 0012                 LL=2*M-N+1
 0013                 LN=N+1
 0014                 LR=M-N+1
 0015                 DO 4 K=1,LL
 0016                 DO 4 J=1,M
 0017                 MS(K)=K+100
 0018               4 A(K,J)=0.0
 0019                 CALL AUTO(N)
 0020                 CALL CROSS(M,N)
 0021                 DO 36 K=1,LR
 0022                 DO 36 J=1,M
 0023                 LN=J-K
 0024                 LNN=IABS(LN)+1
 0025                 IF(LNN-N)35,35,34
 0026              34 A(K,J)=0.0
 0027                 GO TO 36
 0028              35 A(K,J)=PHIFF(LNN)
 0029              36 CONTINUE
 0030                 WRITE(6,300)NSTORM
 0031             300 FORMAT(1H1,I6)
 0032                 WRITE(9,59)
 0033              59 FORMAT(18HNAME              DATA)
 0034                 WRITE(9,60)
 0035              60 FORMAT('ROWS')
 0036                 DO 65 I=1,M
 0037                 WRITE(9,64)MS(I)
 0038              64 FORMAT(1X,'E',2X,'J',I3)
 0039              65 CONTINUE
 0040                 WRITE(9,66)
 0041              66 FORMAT(1X,'N',2X,'Z')
 0042                 WRITE(9,67)
 0043              67 FORMAT('COLUMNS')
 0044                 JJ=1
 0045                 DO 80 K=1,LL
 0046                 DO 75 J=1,M
 0047                 IF(K-M+N-1)70,70,76
 0048              70 WRITE(9,71)MS(K),MS(J),A(K,J)
 0049              71 FORMAT(4X,'H',I3,6X,'J',I3,T26,F9.4)
 0050              75 CONTINUE
 0051                 GO TO 80
 0052              76 WRITE(9,77)MS(K)
```

```
0053            77 FORMAT(4X,'H',I3,6X,'Z',T29,'1.0')
0054               NSLACK=K+100
0055               WRITE(9,85)NSLACK,MS(JJ)
0056            85 FORMAT(4X,'H',I3,6X,'J',I3,T29,'1.0')
0057               JJ=JJ+1
0058            80 CONTINUF
0059               WRITE(9,90)
0060            90 FORMAT('RHS')
0061               DO 95 J=1,M
0062               WRITE(9,92)MS(J),PHIFG(J)
0063            92 FORMAT(4X,'HVECTOR',3X,'J',I3,T25,F9.4)
0064            95 CONTINUE
0065               WRITE(9,100)
0066           100 FORMAT('BOUNDS')
0067               DO 110 J=1,LL
0068               WRITE(9,105)MS(J)
0069           105 FORMAT(' LO LIMIT',T15,'H',I3,T29,'0.0')
0070           110 CONTINUE
0071               WRITE(9,115)
0072           115 FORMAT('ENDATA')
0073               GO TO 1
0074           120 CONTINUE
0075               STOP
0076               END
```

```
FORTRAN IV G LEVEL 1, MOD 3          AUTO          DATE = 70084        09/48/03          PAGE 0001

              SUBROUTINE AUTO(N)
              COMMON F(45),G(80),PHIFF(45),PHIFG(80)
              LAG=0
              DO 30 J=1,N
              SUM = 0.0
              DO 25 I=1,N
              II=I-LAG
              IF(II)25,25,24
           24 SUM=SUM+F(I)*F(II)
           25 CONTINUE
              PHIFF(J)=SUM
           30 LAG=LAG+1
              RETURN
              END
```

```
FORTRAN IV G LEVEL 1, MOD 3          CROSS         DATE = 70084        09/48/03          PAGE 0001

              SUBROUTINE CROSS(M,N)
              COMMON F(45),G(80),PHIFF(45),PHIFG(80)
              LC=0
              DO 50 J=1,M
              TOTAL=0.0
              DO 47 I=1,M
              LS=I-LC
              IF(LS)47,47,45
           45 IF(LS-N)46,46,47
           46 TOTAL=TOTAL+F(LS)*G(I)
           47 CONTINUE
              PHIFG(J)=TOTAL
           50 LC=LC+1
              RETURN
              END
              EXECUTOR.    MPS/360 V2-M6
                                                                                        PAGE     11  -  70/083
SECTION 2 - COLUMNS

NUMBER  .COLUMN.  AT  ...ACTIVITY...  ..INPUT COST..  ..LOWER LIMIT.  ..UPPER LIMIT.  .REDUCED COST.

    14  H101      BS      20.00000           .               .             NONE              .
    15  H102      BS     216.00000           .               .             NONE              .
    16  H103      BS      56.80000           .               .             NONE              .
    17  H104      BS      20.00000           .               .             NONE              .
    18  H105      BS      11.40000           .               .             NONE              .
    19  H106      BS       8.80000           .               .             NONE              .
    20  H107      BS       6.80000           .               .             NONE              .
    21  H108      BS       6.00000           .               .             NONE              .
    22  H109      BS       5.20000           .               .             NONE              .
    23  H110      BS       4.40000           .               .             NONE              .
    24  H111      BS       4.20000           .               .             NONE              .
    25  H112      BS       4.00000           .               .             NONE              .
    26  H113      LL          .            1.00000           .             NONE           1.00000
    27  H114      LL          .            1.00000           .             NONE           1.00000
    28  H115      LL          .            1.00000           .             NONE           1.00000
    29  H116      LL          .            1.00000           .             NONE           1.00000
    30  H117      LL          .            1.00000           .             NONE           1.00000
    31  H118      LL          .            1.00000           .             NONE           1.00000
    32  H119      LL          .            1.00000           .             NONE           1.00000
    33  H120      LL          .            1.00000           .             NONE           1.00000
    34  H121      LL          .            1.00000           .             NONE           1.00000
    35  H122      LL          .            1.00000           .             NONE           1.00000
    36  H123      LL          .            1.00000           .             NONE           1.00000
    37  H124      LL          .            1.00000           .             NONE           1.00000
```

SUMMARY OF DATA FROM OPERATING REPORTS FOR
SOUTH RIVER – OLD PLANT (TRICKLING FILTER) – ATLANTA, GEORGIA

	1968 December	1969 January	February	March	April	May	June	July	August	September	October	November
Adjusted Influent, MGD	3.90	4.50	3.90	4.40	4.80	5.90	4.90	5.10	5.30	4.80	5.10	3.80
Treated Influent, MGD	3.60	4.30	3.80	4.30	4.60	5.10	4.70	4.90	5.20	4.70	5.00	3.80
Bypass Volume, MGD	.30	.20	.10	.10	.20	.80	.20	.20	.10	.10	.10	.00
Bypass Hours in Month	60.75	33.25	18.25	19.50	24.00	98.00	26.50	34.00	11.75	13.75	30.00	9.00
TSS, Avg Influent, ppm	424.00	350.00	196.00	184.00	208.00	194.00	257.00	232.00	250.00	257.00	295.00	232.00
TSS, Avg, lbs	12,730.17	12,551.70	6,211.63	6,598.60	7,979.71	8,251.59	10,073.88	9,480.91	10,842.00	10,073.88	12,301.50	7,352.54
TSS, Avg Plant Effluent, ppm	60.00	80.00	53.00	80.00	65.00	84.00	27.00	60.00	53.00	62.00	65.00	57.00
TSS, Avg Removed Daily, lbs	10,928.73	9,682.74	4,531.95	3,729.64	5,486.05	4,678.73	9,015.53	7,028.95	8,543.49	7,643.60	9,591.00	5,546.09
TSS, Avg Removed Daily, Percent	85.80	77.10	72.90	56.50	68.70	56.70	89.40	74.10	78.70	75.80	77.90	75.40
TSS, Avg to Stream, Daily, lbs.	2,862.28	3,452.76	1,843.14	3,022.41	2,840.60	4,867.22	1,487.02	2,838.93	2,507.01	2,644.61	2,956.53	1,806.45
BOD, Avg Daily Influent, ppm	335.00	211.00	206.00	281.00	175.00	160.00	305.00	225.00	208.00	261.00	177.00	301.00
BOD, Avg Daily Influent, lbs	10,058.04	7,566.88	6,528.55	10,077.22	6,713.70	6,805.44	11,955.39	9,194.85	9,020.54	10,230.67	7,380.90	9,539.29
BOD, Avg Plant Effluent, ppm	43.00	55.00	32.00	40.00	31.00	34.00	31.00	34.00	34.00	37.00	44.00	58.00
BOD, Avg Removed Daily, lbs	8,767.00	5,594.47	5,514.40	8,642.74	5,524.41	5,359.28	10,740.25	7,805.40	7,546.02	8,780.34	5,546.10	7,701.15
BOD, Avg Removed Daily, Percent	87.10	73.90	84.40	85.70	82.20	78.70	89.80	84.80	83.60	85.80	75.10	80.70
BOD, Avg to Stream Daily, lbs	2,129.21	2,324.35	1,185.95	1,668.83	1,481.19	2,513.68	1,723.88	1,764.75	1,647.99	1,668.00	1,982.41	1,838.14
Average Screenings, cf/day	31.00	33.00	38.50	30.00	28.00	32.00	27.00	24.00	29.00	7.00	8.00	11.00
Average Screenings, cf/MG	8.61	7.67	10.13	6.97	6.08	6.27	5.74	4.89	5.57	1.48	1.60	2.89
Average Grit Removed, cf/day	61.00	63.00	71.40	55.00	63.00	52.00	61.00	60.00	74.00	10.00	20.00	12.00
Average Grit Removed, cf/MG	16.94	14.65	18.78	12.79	13.69	10.19	12.97	12.24	14.23	2.12	4.00	3.15
Grit Organics	31.00	--	--	--	--	--	--	--	--	--	--	--

SUMMARY OF DATA FROM OPERATING REPORTS FOR
SOUTH RIVER – NEW PLANT (AERATION) – ATLANTA, GEORGIA

	1968 December	1969 January	February	March	April	May	June	July	August	September	October	November
Adjusted Influent, MGD	10.30	8.90	7.80	9.40	12.00	14.10	10.90	11.10	12.40	9.80	10.80	11.20
Treated Influent, MGD	10.10	8.90	7.50	9.00	11.70	13.10	10.80	10.90	12.10	9.70	10.70	11.00
Bypass Volume, MGD	.20	--	.30	.40	.30	1.00	.10	.20	.30	.10	.10	.20
Bypass Hours in Month	10.25	1.00	18.25	19.50	24.00	34.50	5.00	10.75	11.75	4.75	3.00	9.00
TSS, Avg Influent, ppm	416.00	350.00	196.00	184.00	208.00	194.00	257.00	232.00	250.00	257.00	295.00	232.00
TSS, Avg, lbs	35,041.34	25,979.10	12,259.80	13,811.04	20,296.22	21,195.27	23,148.50	21,090.19	25,228.50	20,790.78	26,325.21	21,283.68
TSS, Avg Plant Effluent, ppm	70.00	83.00	67.00	66.00	49.00	77.00	66.00	76.00	56.00	54.00	70.00	82.00
TSS, Avg Removed Daily, lbs	29,144.96	19,818.34	8,068.95	8,857.08	15,514.89	12,782.71	17,203.74	14,181.33	19,577.31	16,422.28	20,078.55	13,761.00
TSS, Avg Removed Daily, Percent	83.10	76.20	65.80	64.10	76.40	60.30	74.30	67.20	77.50	78.90	76.20	64.60
TSS, Avg To Stream Daily, lbs	6,590.26	6,160.76	4,681.24	5,567.78	5,301.74	10,030.52	6,159.09	7,295.83	6,276.69	4,582.83	6,492.69	7,909.65
BOD, Avg Daily Influent, ppm	299.00	211.00	206.00	282.00	175.00	160.00	305.00	225.00	208.00	261.00	177.00	301.00
BOD, Avg Daily Influent, lbs	25,185.96	15,661.68	12,885.30	21,166.92	17,076.15	17,480.64	27,471.96	20,453.85	20,990.11	21,114.37	15,795.12	27,613.74
BOD, Avg Plant Effluent, ppm	57.00	38.00	57.00	43.00	35.00	40.00	36.00	34.00	30.00	22.00	35.00	52.00
BOD, Avg Removed Daily, lbs	20,384.62	12,841.09	9,319.95	17,939.34	13,660.92	13,110,48	24,229.36	17,363.04	17,962.69	19,334.61	12,671.79	22,843.26
BOD, Avg Removed Daily, Percent	80.90	81.90	72.30	84.70	80.00	75.00	88.10	84.80	85.50	91.50	80.20	82.70
BOD, Avg to Stream Daily, lbs	5,300.07	2,820.59	4,080.76	4,168.63	3,853.08	5,704.56	3,496.97	3,466.11	3,547.83	1,997.40	3,270.94	5,272.54
Average Screenings, cf/day	31.00	33.00	38.50	30.00	28.00	32.00	27.00	24.00	29.00	15.00	18.00	19.00
Average Screenings, cf/MG	3.06	3.70	5.13	3.33	2.39	2.44	2.50	2.20	2.39	1.54	1.68	1.72
Average Grit Removed, cf/day	61.00	63.00	71.40	55.00	63.00	52.00	61.00	60.00	74.00	34.00	44.00	32.00
Average Grit Removed, cf/MG	6.03	7.07	9.52	6.11	5.38	3.96	5.64	5.50	8.11	3.50	4.11	2.90
Grit Organics	31.00	--	--	--	--	--	--	--	--	--	--	--

SUMMARY OF DATA FROM OPERATING REPORTS FOR
INTRENCHMENT CREEK WASTEWATER TREATMENT PLANT, SOUTH RIVER BASIN – ATLANTA, GEORGIA

	1968 December	1969 January	February	March	April	May	June	July	August	September	October	November
Adjusted Influent, MGD	13.30	13.50	14.20	13.70	14.40	14.70	13.80	12.90	13.00	12.40	11.60	12.10
Treated Influent, MGD	12.60	12.80	13.30	12.90	13.60	13.90	13.40	12.30	12.50	11.80	11.30	11.70
Bypass Volume, MGD	.70	.70	.90	.80	.80	.80	.40	.60	.50	.60	.30	.40
Bypass Hours in Month	63.00	42.40	64.50	62.50	72.50	74.50	30.00	47.50	46.00	35.50	25.00	26.00
TSS, Avg Influent, ppm	266.00	167.00	195.00	207.00	287.00	209.00	215.00	308.00	207.00	354.00	321.00	205.00
TSS, Avg, lbs	27,952.34	17,827.58	21,629.79	22,270.30	32,552.68	24,228.53	24,027.54	31,595.25	21,579.75	34,837.84	30,251.68	20,003.49
TSS, Avg Plant Effluent, ppm	61.00	73.00	72.00	96.00	89.00	54.00	53.00	47.00	53.00	66.00	70.00	47.00
TSS, Avg Removed Daily, lbs	21,542.21	10,034.68	13,643.40	11,942.04	22,457.94	17,968.52	18,104.47	26,773.89	16,054.50	28,342.64	23,654.74	15,417.32
TSS, Avg Removed Daily, Percent	77.00	56.20	63.00	53.60	68.90	74.10	75.30	84.70	74.30	81.30	78.10	77.00
TSS, Avg to Stream Daily, lbs	7,963.03	8,767.84	9,450.06	11,709.36	12,009.60	7,654.45	6,640.31	6,362.59	6,388.44	8,266.61	7,400.08	5,270.05
BOD, Avg Daily Influent, ppm	352.00	350.00	388.00	379.00	346.00	386.00	382.00	343.00	343.00	374.00	387.00	384.00
BOD, Avg Daily Influent, lbs	36,989.56	37,363.20	43,037.73	40,775.09	39,244.70	44,747.43	42,690.79	35,185.62	35,757.75	36,806.08	36,471.65	37,469.95
BOD, Avg Plant Effluent, ppm	52.00	78.00	109.00	113.00	91.00	52.00	39.00	30.00	44.00	36.00	47.00	63.00
BOD, Avg Removed Daily, lbs	31,525.19	29,036.54	30,947.23	28,617.87	28,923.11	38,719.27	38,332.30	32,108.16	31,170.75	33,263.24	32,042.27	31,322.53
BOD, Avg Removed Daily, Percent	85.20	77.70	71.90	70.10	73.60	86.50	89.70	91.20	87.10	90.30	87.80	83.50
BOD, Avg to Stream Daily, lbs	7,519.34	10,369.96	15,002.82	14,685.90	12,630.10	8,603.55	5,632.84	4,793.83	6,017.31	7,516.03	5,397.65	7,428.44
Average Screenings, cf/day	27.90	18.20	27.96	23.52	31.05	34.84	18.90	15.68	21.77	17.10	10.88	9.90
Average Screenings, cf/MG	2.21	1.42	2.10	1.82	2.28	2.50	1.41	1.27	1.74	1.44	.96	.84
Average Grit Removed, cf/day	67.90	74.47	65.57	52.69	76.50	108.44	67.50	78.82	91.89	89.10	88.40	77.85
Average Grit Removed, cf/MG	5.38	5.81	4.93	4.08	5.62	7.80	5.03	6.40	7.35	7.55	7.82	6.65
Grit Organics	33.00	18.00	30.00	18.90	26.00	3.33	26.50	16.00	7.00	17.00	14.00	27.00

SUMMARY OF OPERATING DATA

SNAPFINGER CREEK WPC PLANT – SOUTH RIVER BASIN, DEKALB COUNTY, GEORGIA

	1969 January	February	March	April	May	June	July	August	September	October	November	December
Average Flow, MGD	2.33	2.61	2.66	2.52	2.76	2.54	1.79	2.09	1.97	2.53	2.52	3.59
Bypass hours in month	34	20	18	60	52	0	4	8	30	78	144	168
TSS Avg Influent, ppm	119	122	64	76	70	98	108	132	192	71	82	96
TSS Avg Effluent, ppm	44	42	32	29	20	21	21	39	130	31	22	21
TSS Removed Daily, %	63	66	50	62	71	79	81	70	32	56	73	78
TSS to Stream Daily, lbs*	1,130	963	727	692	540	450	320	697	2,172	1,409	714	1,134
BOD, Avg Influent, ppm	141	174	124	132	73	97	50	123	46	99	86	107
BOD, Avg Effluent, ppm	76	51	45	33	18	27	47	35	28	42	29	37
BOD, Removed Daily, %	46	71	64	75	75	72	6	72	39	58	66	65
BOD to Stream Daily, lbs*	1,880	1,094	1,042	865	502	572	700	627	472	1,963	1,850	1,579

* Adjusted according to bypass hours in a month

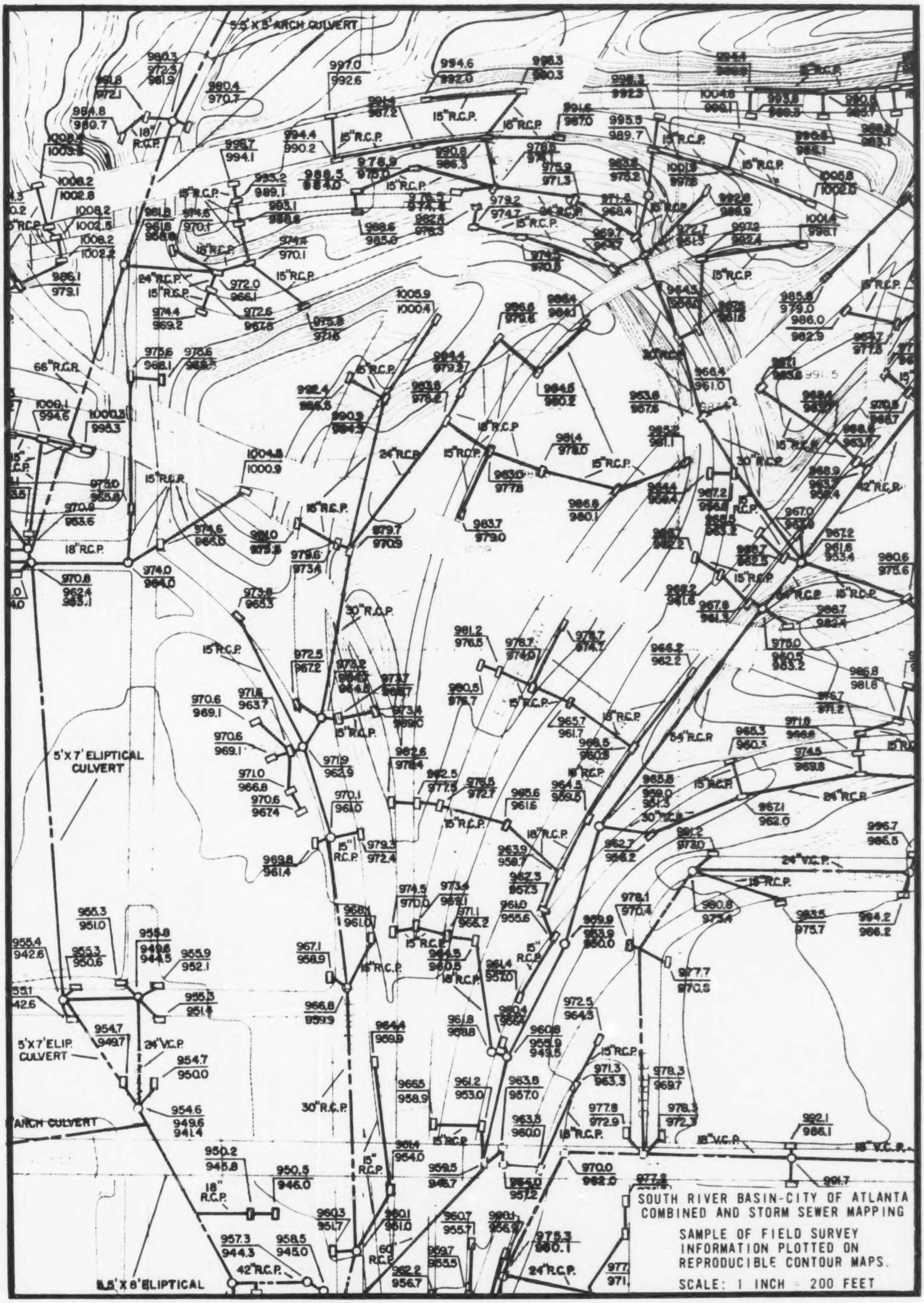
SUMMARY OF OPERATING DATA

SHOAL CREEK WASTEWATER TREATMENT PLANT

SOUTH RIVER BASIN, DEKALB COUNTY, GEORGIA

	1969 January	February	March	April	May	June	July	August	September	October	November	December
Average Flow, MGD	2.86	3.20	2.76	2.79	2.49	2.87	2.74	2.77	2.84	2.64	2.64	2.76
Bypass hours in month	16	120	0	48	40	0	28	24	0	0	14	0
TSS Avg Influent, ppm	132	80	83	95	119	125	123	125	191	183	63	45
TSS Avg Effluent, ppm	25	24	26	13	30	13	21	19	60	14	13	22
TSS Removed Daily, %	81	70	69	86	75	90	83	85	69	92	79	51
TSS to Stream Daily, lbs*	630	905	598	437	723	311	567	518	1,420	308	844	506
BOD Avg Influent, ppm	182	217	192	173	137	140	52	129	113	130	127	186
BOD Avg Effluent, ppm	101	43	36	30	28	34	15	19	20	29	39	45
BOD Removed Daily, %	45	80	81	83	80	76	71	85	82	78	69	76
BOD to Stream Daily, lbs*	2,362	1,974	828	919	703	814	374	521	473	638	897	1,035

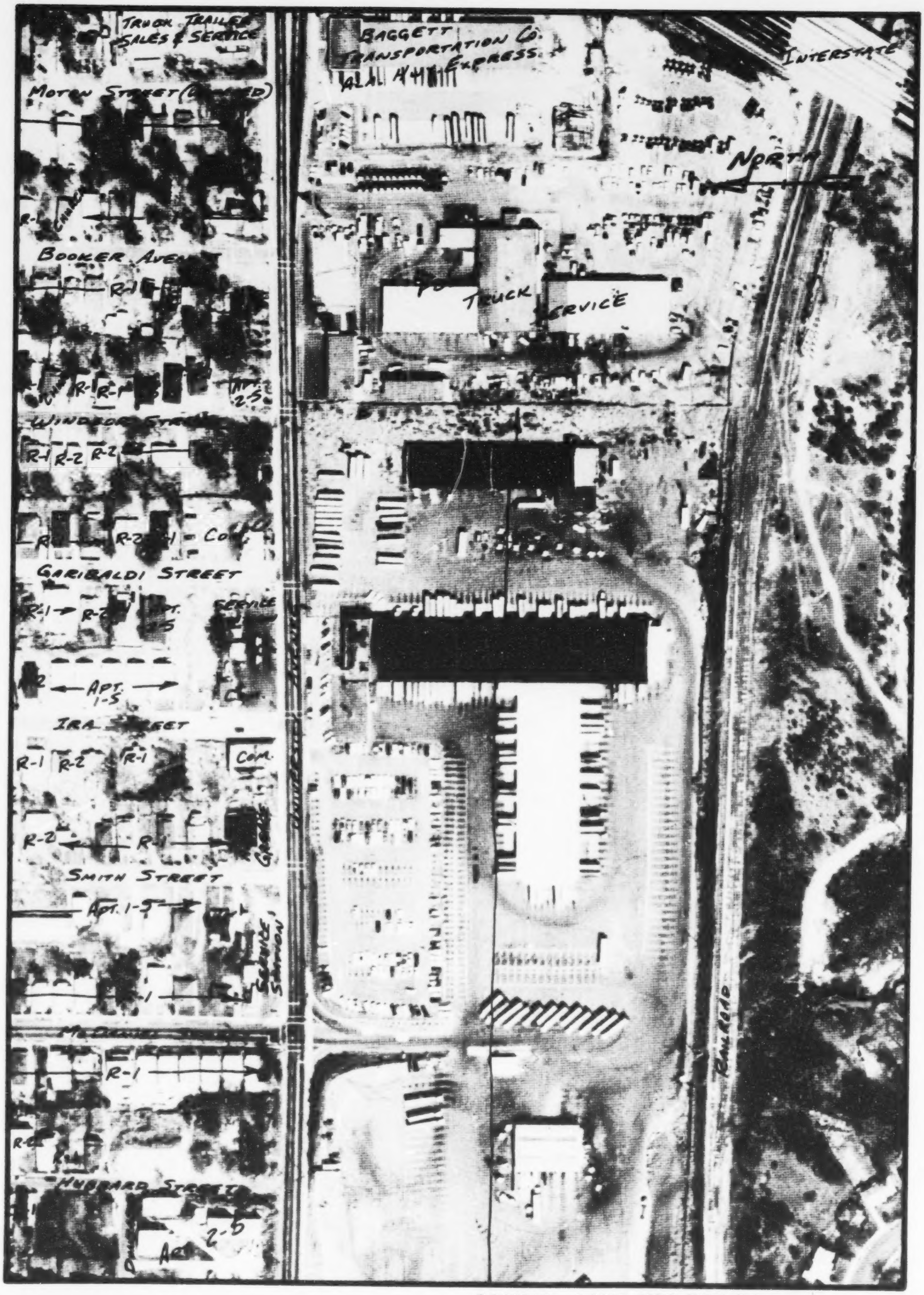
* Adjusted according to bypass hours in month


5.5' X 5' ARCH CULVERT
66" R.C.P.
5'X7' ELIPTICAL CULVERT
5'X7' ELIP. CULVERT
24" V.C.P.
ARCH CULVERT
5.5' X 8' ELIPTICAL
42" R.C.P.
30" R.C.P.
18" V.C.P.
24" V.C.P.
54" R.C.P.
30" R.C.P.
24" R.C.P.
18" R.C.P.
15" R.C.P.
60" R.C.P.

SOUTH RIVER BASIN-CITY OF ATLANTA
COMBINED AND STORM SEWER MAPPING

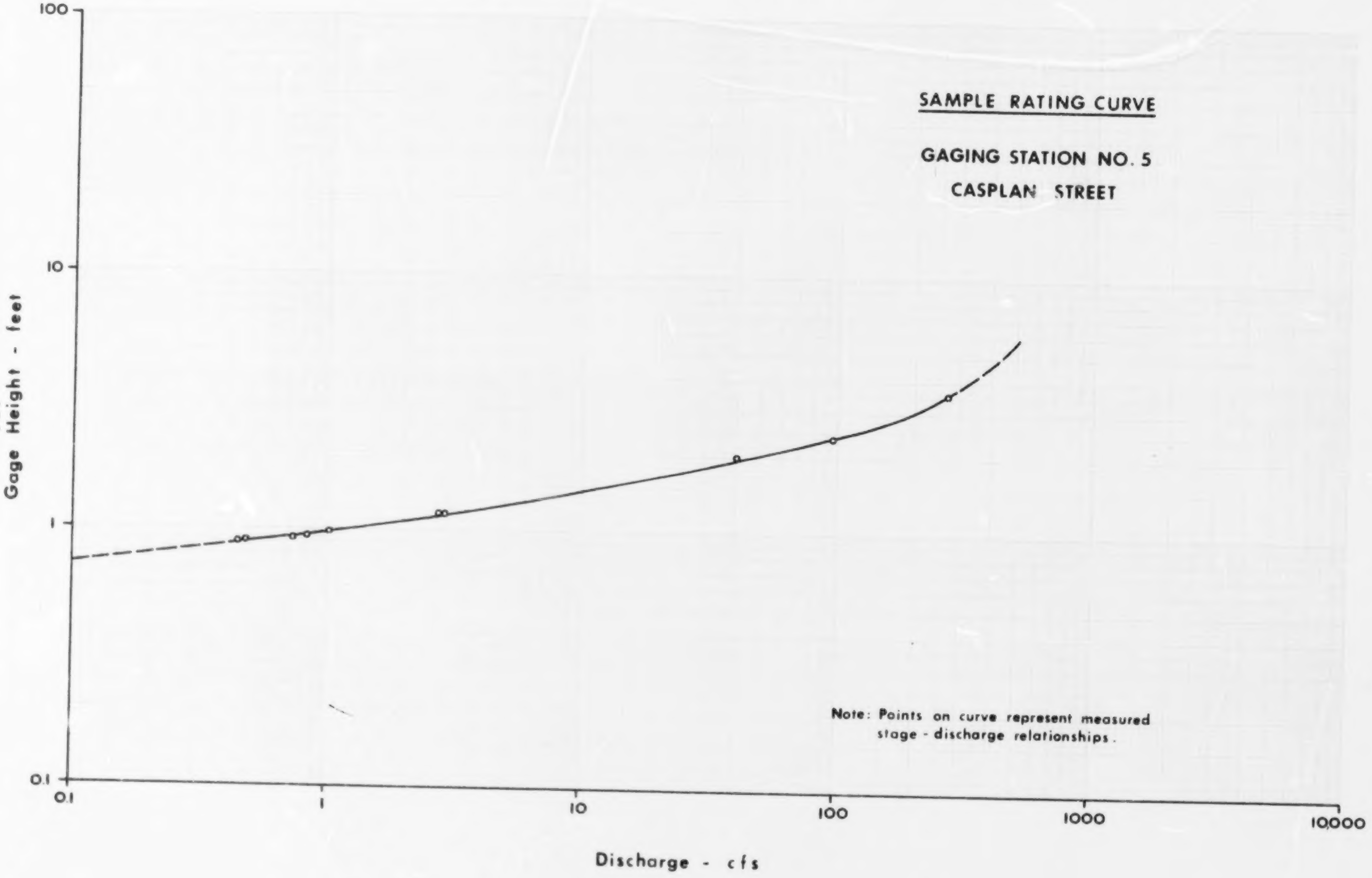
SAMPLE OF FIELD SURVEY
INFORMATION PLOTTED ON
REPRODUCIBLE CONTOUR MAPS.

SCALE: 1 INCH = 200 FEET


TRUCK TRAILER SALES & SERVICE
BAGGETT TRANSPORTATION CO.
EXPRESS
INTERSTATE
MOTEN STREET (CLOSED)
NORTH
R-1
CHURCH
BOOKER AVE.
R-1
TRUCK SERVICE
R-1
R-1
APT. 2-5
WINDSOR STREET
R-1
R-2
R-2
R-2
R-1 = COM.
GARIBALDI STREET
R-1
R-2
APT. 1-5
SERVICE
R-2
APT. 1-5
COM.
IRA STREET
R-1
R-2
R-1
COM.
R-2
R-1
GARAGE
SMITH STREET
APT. 1-5
SERVICE STATION
R-1
McDANIEL
R-1
R-2
R-1
HUBBARD STREET
APT. 2-5
RAILROAD

SAMPLE - LAND USE SURVEY

Drainage Area No. 3, McDaniel St.



SAMPLE RATING CURVE

GAGING STATION NO. 5
CASPLAN STREET

Note: Points on curve represent measured stage - discharge relationships.

APPENDIX

SUBBASIN NO. 3

McDaniel Street Combined Sewer Area

Survey of Possible Sources of Industrial or Trade Waste Discharges (Grouped by Sheet Numbers, Aerial Map)

Sheet P – 16

1. Electric Storage Battery Company "EXIDE"
2. Great Dane Trailers, Inc., Southeastern Leasing
3. Pollack Paper Company (a)
4. Armour and Company
5. Southern Paint Products Manufacturers
6. Link – Belt
7. Cotton Waste Railway Supply & Manufacturers Co.
8. Reliable Truck Repair (Typical)
9. Johnson Manufacturing
10. Service Stations (Typical)
11. Coin Laundries and Dry Cleaners (Typical)

Sheet P – 17

12. Refrigeration Sales & Service (Typical)
13. Tire Recapping (Typical)
14. Palmour Coffee Company
15. Steam Specialty Company
16. Produce Market (Typical)
17. Auto Junk Yard
18. Auto Electric Exchange
19. Wise Potato Chips
20. Seed Company (Typical)
21. Globe Products Company (b)
22. Harry Alter Company
23. McQuay – Norris Manufacturing
24. Reading Tube Corporation
25. Southeastern Industries
26. Metals Company
27. General Electric
28. Southeastern Warehouse
29. Hudson Manufacturing Company
30. Lancaster Products Company
31. Pillsbury
32. R. C. Can Company
33. Joslyn Manufacturing
34. Precisionaire
35. Newell Manufacturing
36. Dixie Industrial Finishing Co. (Electroplating)
37. Electric Company (Typical)
38. Merita Bakery (d)
39. Ruralist Press
40. Henson Bakery Supplies
41. Cullen Adams (Upholstry)
42. Brown Metal Works
43. Engraving Company
44. Carpenter Steel Company
45. Atlantic Ice Company

46. Berkeley Pump
47. Envelope Manufacturer
48. Abrams Fixture Corporation

Sheet P – 18

49. Fruehauf Trailers
50. Harris Rim & Wheel Inc. Service
51. Printing Company
52. Kingston Printing
53. McDaniel Mattress Company
54. Auto Truck Electric Company
55. Gas Company

Sheet Q – 16

56. Atlanta Intercel
57. Ryder Truck Lines
58. Ford Truck Service
59. Baggett Transportation Company
60. Foy Brick Company
61. Miller Lumber Company
62. Southern Lead Burning Company
63. Dell Industries

Sheet Q – 17

64. Central Neon
65. London Iron & Metal Company
66. Metal Stamping
67. W.C. Caye, Construction Equipment
68. Southern Railway
69. Famous Foods (Processor)
70. Tractor and Machinery Company
71. G. E. Warehouse
72. Building Material Supply
73. Valiant Steel
74. Wadsworth Electric Manufacturing Company
75. Superior Bumper Products
76. Spinks Scale Company
77. New Dixie Trucking Company
78. Peter Pan Baking Company
79. Sugar & Spice Bakery
80. Lundsford Wilson Company
81. White House Food Products
82. National Fruit Product Company
83. Food Distributors
84. Industrial Chemical Manufacturers
85. American Associated Companies
86. Dry Ice Company (e)
87. J & B Smith Company (Reconditioning Steel Drums)
88. Wholesale Florists
89. Kimbro Warehouse
90. Cary Chapman Co. (Electrical & Electronic Products)
91. Gordy Tire Company
92. Woffard Cabinet Company
93. Commercial Solvents Corporation
94. Southern Sales and Warehouse Company
95. Johnson–Fluker Company
96. Industrial Equipment Company
97. Speed Check Company
98. Industrial Door, Inc.
99. Quality Used Tire Company

Sheet Q – 18

100. Southern Mills
101. Swift & Company Oleomargarine Refinery
102. Metals, Inc.
103. Barrel–Town
104. International Harvester
105. Dixie Rubber Company
106. Select Foods
107. Leaseway of Georgia, Inc.
108. Atlanta Betting–Republic Rubber

109. Atlas Supply Company
110. American Excelsor Corporation
111. General Lithographing
112. William Bishop, Electrical Manufacturing Agent
113. Alterman Brothers
114. Thomas Paint Manufacturing Company
115. Joana – Western Mills
116. John Underwood Company
117. Rubber Printing Plates
118. Fred K. H. Levy Company (Inks)
119. Bumper Service of Atlanta
120. New Method Laundry

Notes:

(a) Solvents, BOD 832, pH 11.1

(b) Solvents

(c) pH 3.5, can also be high

(d) BOD 1112, Suspended Solids 206

(e) Low pH

```
STATION  2 STARTING 136/ 3  0

    STA. DAY HR MN    GAGE      FLOW          FLOW GRAPH                                       500    TOTAL CF
==============================================================================================================
    2  136/ 3  0      0.38       0.0          I                                                             0.
    2  136/ 3 15      1.44     87.00          I*********                                                39150.
    2  136/ 3 30      1.30     63.00          I******                                                  106650.
    2  136/ 3 45      1.00     25.00          I***                                                     146250.
    2  136/ 4  0      1.25     55.00          I******                                                  182250.
    2  136/ 4 15      0.60      2.50          I                                                        208125.
    2  136/ 4 30      0.38       0.0          I                                                        209250.
    2  136/ 4 45      0.38       0.0          I                                                        209250.
    2  136/ 5  0      0.38       0.0          I                                                        209250.
    2  136/ 5 15      0.39       0.0          I                                                        209250.
    2  136/ 5 30      0.38       0.0          I                                                        209250.
    2  136/ 5 45      0.38       0.0          I                                                        209250.
    2  136/ 6  0      0.38       0.0          I                                                        209250.
    2  136/ 6 15      0.38       0.0          I                                                        209250.
    2  136/ 6 30      0.38       0.0          I                                                        209250.
    2  136/ 6 45      1.25     55.00          I******                                                  234000.
    2  136/ 7  0      2.07    226.30          I***********************                                 360585.
    2  136/ 7 15      2.36    281.80          I****************************                            589230.
    2  136/ 7 30      2.04    220.60          I**********************                                  815309.
    2  136/ 7 45      1.72    150.00          I***************                                         982079.
    2  136/ 8  0      1.50     97.00          I**********                                             1093229.
    2  136/ 8 15      1.30     63.00          I******                                                 1165229.
    2  136/ 8 30      1.64    130.00          I*************                                          1252079.
    2  136/ 8 45      1.42     83.00          I********                                               1347929.
    2  136/ 9  0      1.16     42.50          I****                                                   1404404.
    2  136/ 9 15      1.08     33.00          I***                                                    1438379.
    2  136/ 9 30      1.08     33.00          I***                                                    1468079.
    2  136/ 9 45      1.10     35.50          I****                                                   1498904.
    2  136/10  0      0.98     24.00          I**                                                     1525679.
    2  136/10 15      0.82     11.00          I*                                                      1541429.
    2  136/10 30      0.65      4.00          I                                                       1548179.
    2  136/10 45      0.95     21.00          I**                                                     1559429.
    2  136/11  0      1.14     41.00          I****                                                   1587329.
    2  136/11 15      0.75      7.50          I*                                                      1609154.
    2  136/11 30      0.52      0.83          I                                                       1612902.
    2  136/11 45      0.38       0.0          I                                                       1613275.
    2  136/12  0      0.38       0.0          I                                                       1613275.
    2  136/12 15      0.38       0.0          I                                                       1613275.
    2  136/12 30      0.38       0.0          I                                                       1613275.
    2  136/12 45      0.38       0.0          I                                                       1613275.
    2  136/13  0      0.39       0.0          I                                                       1613275.
    2  136/13 15      0.38       0.0          I                                                       1613275.
    2  136/13 30      0.38       0.0          I                                                       1613275.
    2  136/13 45      0.38       0.0          I                                                       1613275.
    2  136/14  0      0.38       0.0          I                                                       1613275.
    2  136/14 15      0.81     10.50          I*                                                      1618000.
    2  136/14 30      0.96     22.00          I**                                                     1632625.
    2  136/14 45      0.69      5.20          I*                                                      1644864.
    2  136/15  0      0.43      0.05          I                                                       1647226.
    2  136/15 15      0.38       0.0          I                                                       1647248.
```

```
STATION  3 STARTING 136/ 2 45

  STA. DAY HR MN      GAGE      FLOW               FLOW GRAPH                                                      250         TOTAL CF
=======================================================================================================================================
    3  136/ 2 45      0.31       0.0               I                                                                            0.
    3  136/ 3  0      1.13      32.00              I••••••                                                                  14400.
    3  136/ 3 15      0.90      14.50              I•••                                                                     35325.
    3  136/ 3 30      0.64       3.00              I•                                                                       43200.
    3  136/ 3 45      0.51       0.90              I                                                                        44955.
    3  136/ 4  0      0.44       0.24              I                                                                        45468.
    3  136/ 4 15      0.41       0.08              I                                                                        45612.
    3  136/ 4 30      0.38       0.01              I                                                                        45652.
    3  136/ 4 45      0.37       0.0               I                                                                        45657.
    3  136/ 5  0      0.36       0.0               I                                                                        45657.
    3  136/ 5 15      0.35       0.0               I                                                                        45657.
    3  136/ 5 30      0.34       0.0               I                                                                        45657.
    3  136/ 5 45      0.33       0.0               I                                                                        45657.
    3  136/ 6  0      0.33       0.0               I                                                                        45657.
    3  136/ 6 15      0.34       0.0               I                                                                        45657.
    3  136/ 6 30      1.20      39.00              I••••••••                                                                63207.
    3  136/ 6 45      1.38      62.20              I••••••••••••                                                           108747.
    3  136/ 7  0      1.76     124.80              I•••••••••••••••••••••••••                                              192897.
    3  136/ 7 15      1.61      96.80              I•••••••••••••••••••                                                    292617.
    3  136/ 7 30      1.30      52.00              I••••••••••                                                             359577.
    3  136/ 7 45      1.02      23.00              I•••••                                                                  393327.
    3  136/ 8  0      1.20      39.00              I••••••••                                                               421227.
    3  136/ 8 15      1.50      80.00              I••••••••••••••••                                                       474777.
    3  136/ 8 30      1.20      39.00              I••••••••                                                               528327.
    3  136/ 8 45      1.00      21.00              I••••                                                                   555327.
    3  136/ 9  0      0.88      13.50              I•••                                                                    570852.
    3  136/ 9 15      0.87      12.70              I•••                                                                    582642.
    3  136/ 9 30      0.90      14.50              I•••                                                                    594882.
    3  136/ 9 45      0.90      14.50              I•••                                                                    607932.
    3  136/10  0      0.80       8.80              I••                                                                     618416.
    3  136/10 15      0.76       7.00              I•                                                                      625526.
    3  136/10 30      0.70       4.70              I•                                                                      630791.
    3  136/10 45      0.85      11.40              I••                                                                     638036.
    3  136/11  0      0.80       8.80              I••                                                                     647126.
    3  136/11 15      0.65       3.30              I•                                                                      652571.
    3  136/11 30      0.55       1.35              I                                                                       654664.
    3  136/11 45      0.50       0.77              I                                                                       655618.
    3  136/12  0      0.49       0.65              I                                                                       656256.
    3  136/12 15      0.48       0.58              I                                                                       656810.
    3  136/12 30      0.47       0.46              I                                                                       657278.
    3  136/12 45      0.46       0.38              I                                                                       657656.
    3  136/13  0      0.45       0.30              I                                                                       657962.
    3  136/13 15      0.44       0.24              I                                                                       658205.
    3  136/13 30      0.43       0.19              I                                                                       658398.
    3  136/13 45      0.42       0.14              I                                                                       658547.
    3  136/14  0      0.41       0.08              I                                                                       658645.
    3  136/14 15      0.40       0.04              I                                                                       658699.
    3  136/14 30      0.39       0.02              I                                                                       658726.
    3  136/14 45      0.38       0.01              I                                                                       658740.
    3  136/15  0      0.37       0.0               I                                                                       658744.
    3  136/15 15      0.36       0.0               I                                                                       658744.
    3  136/15 30      0.35       0.0               I                                                                       658744.
    3  136/15 45      0.34       0.0               I                                                                       658744.
    3  136/16  0      0.34       0.0               I                                                                       658744.
    3  136/16 15      0.33       0.0               I                                                                       658744.
    3  136/16 30      0.90      14.50              I•••                                                                    665269.
    3  136/16 45      1.16      35.00              I•••••••                                                                687544.
    3  136/17  0      0.86      12.10              I••                                                                     708739.
```

Analyses of Water Quality

Results of water quality analyses were punched on computer cards, and a program was written to collate and print updated records at intervals throughout the study. A sample of the output is included on the following page. Symbols at the top of each column of data are as follows:

Symbols	Description	Unit
BOD	Biochemical Oxygen Demand (5–day, 20° C)	mg/l
COD	Chemical Oxygen Demand	mg/l
DO	Dissolved Oxygen (Axide Modification of Iodometric Method)	mg/l
SST	Total Suspended Solids	mg/l
SSV	Volatile Suspended Solids	mg/l
SSB	Settleable Suspended Solids	mg/l
COND	Conductivity (Industrial Instruments Type RC Conductivity Bridge)	μmhos/cm
PH	pH (Electrometric Method)	---
ACID	Acidity – as $CaCO_3$ (Indicator Method)	mg/l
ALK	Alkalinity – as $CaCO_3$	mg/l
PHOS	Orthophosphate ($SnCl_2$ Method)	mg/l
CHL	Chlorine Demand	mg/l
REST	Total Residue	mg/l
RESV	Volatile Residue	mg/l
COLI	Total Coliform (Membrane Filter)	MPN/100 ml
FCOL	Fecal Coliform (Membrane Filter)	MPN/100 ml
FSTR	Fecal Streptococcus (Membrane Filter)	MPN/100 ml
GAHT	Gage Height	feet

The first line of each pair of lines on the computer printout lists the sample location, date, time and method, and the values obtained for each water quality test. Missing tests are denoted by (–0.0). For instance, the first line of data shown was obtained at station 8 (South River at Bouldercrest Road) on day 87 (of the calendar year 1969), at hour 1012, and by method 0 (automatic sampler). The second line presents loads of some of the parameters in 1,000 lbs/day total bacterial counts, and flow rate corresponding to the stated gage height, in this case 87.80 cfs.

1 *Accession Number*

2 *Subject Field & Group*

05 – B

SELECTED WATER RESOURCES ABSTRACTS
INPUT TRANSACTION FORM

5 *Organization*

BLACK, CROW & EIDSNESS, INC., CONSULTING ENGINEERS
Gainesville, Florida

6 *Title*

Storm and Combined Sewer Pollution Sources and Abatement – Atlanta, Georgia

10 *Author(s)*

Vilaret, Dr. Manuel R.
Pyne, R. David G.

16 *Project Designation*

11024 ELB 04 – 71

21 *Note*

22 *Citation*

23 *Descriptors (Starred First)*

*Overflow, *Storm Runoff, *Wastewater Treatment, *Rainfall–Runoff Relationships, Water Quality, Storage Capacity, Treatment Methods, Treatment Costs, Cost–Benefit Analysis, Simulation, Frequency Analysis, Land–Use Indicators, Pollutants

25 *Identifiers (Starred First)*

*Combined Sewers, Quantity of Overflow, Quality of Overflow.

27 *Abstract*

Six urban drainage basins within the City of Atlanta, Georgia, served by combined and separate sewers, are studied to determine the major pollution sources during storm events. Rainfall frequency analysis and simulation techniques are utilized to obtain design criteria for alternative pollution abatement schemes.

High frequency storms are shown to cause the worst impact and most of the pollution from combined sewer areas. Annual BOD from these areas is 2,078,000 pounds, or 460 lbs/acre, of which 57 percent is due to storms of two–week or higher frequency. Bypassing of wastewater treatment plant flows during storms adds 690,000 pounds BOD/year. Runoff from storm–sewered areas, at 253 lbs/acre, adds 5,577,000 pounds/year. Overflows and bypassed flows have severe impact upon the South River, due to their high deoxygenation rates and coliform concentrations.

Annual BOD reduction from combined sewer areas of 57 percent may be achieved for a total annual cost of $165,000, by modifying the three regulators and treating 80 percent of the overflows, in conjunction with storage sufficient to contain a two–week storm. Alternate, less favorable solutions include storage and treatment at existing treatment plants, and storage with release to receiving streams after chlorination. Separation of combined sewers would achieve 60 percent BOD removal for $3,030,000/year.

Abstractor R. David G. Pyne

Institution Black, Crow & Eidsness, Inc.

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